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consulting engineers, llc.
A Salas O'Brien Company
4700 Lincoln Road NE, Albuquerque, NM 87109
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Structural Calculations
Revised Fit-Up Permit Submittal

Project
Project Sunshine
Auburndale, Florida

Building 1

Project Engineer: **Pietre Drayer, PE, and Austin Rowan, PE**
Engineer of Record: **Nathan Mensay, PE, SE**
DATE: **09/21/2026**

Nathan B Mensay

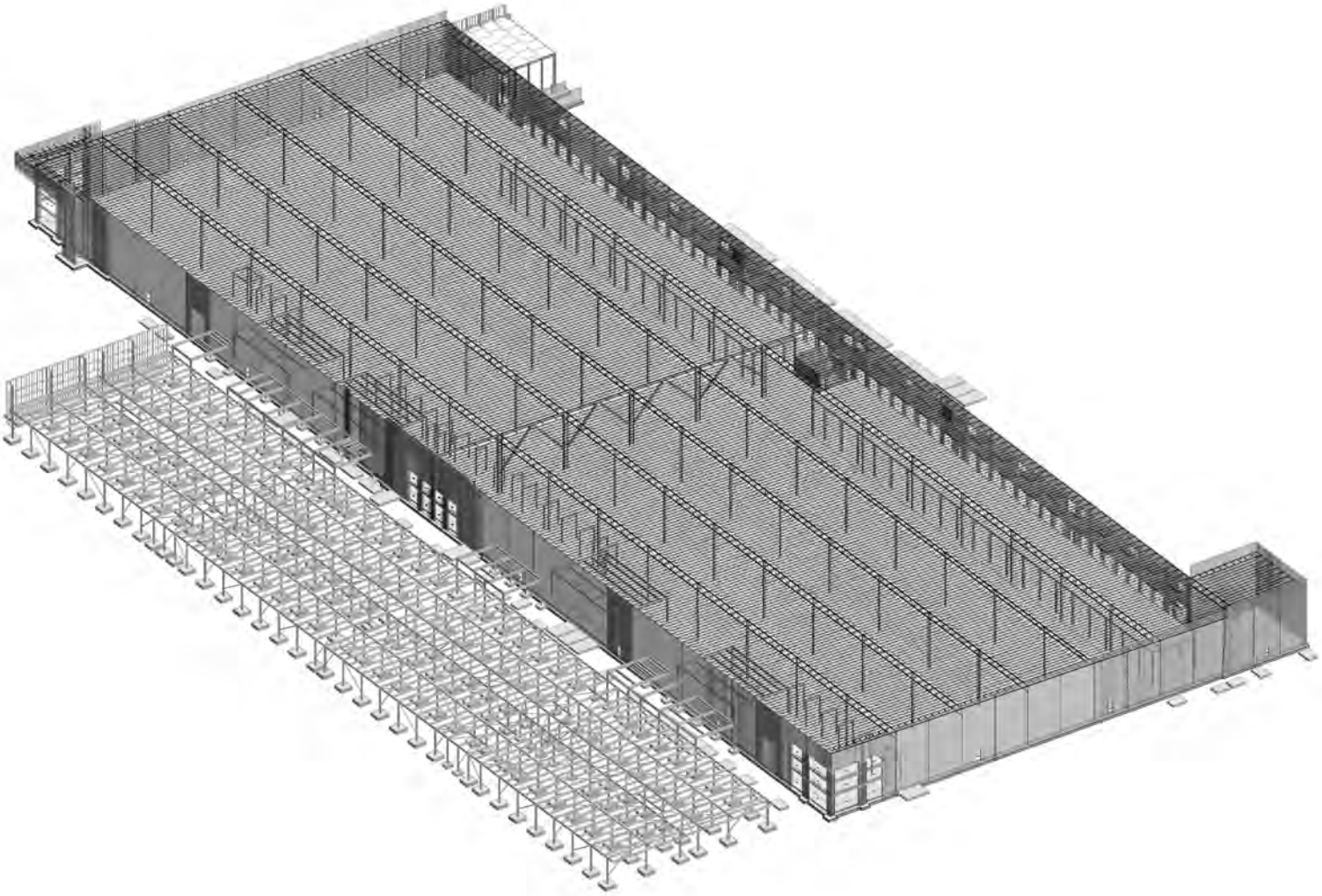
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DN: CN=Nathan B Mensay, dnQualifier=A01410D0000019DB09E756A003AF7CB, O=Unaffiliated, C=US
Reason: Digital Signature
Date: 2026.09.21 22:00:53-06'00'



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Chapter 1 - Loads and Design Criteria



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PHONE: (505) 344-4080

SHEET NO. 1 OF 2
PROJECT Auburndale Data Center
SUBJECT Design Criteria
CLIENT Nuclei JOB NO.
CALCULATED BY PD DATE 3/12/2026

Design Criteria - IBC 2021

Code Criteria

Project Name:	Auburndale Data Center	
Building code:	IBC 2021	References:
Location :	Auburndale, FL	
Building Risk Category	II	[IBC 2021 table 1604.5]
Construction Type:		[IBC 2021 table 601]

Fire Rating Requirements:

Floor Rating:	See Arch	hrs	[IBC 2021 table 601]
Primary Frame:	See Arch	hrs	
Roof Construction	See Arch	hrs	

Live Load Criteria	Uniform LL, psf	Reducible?	Concentrated LL, lbs	Partition Load, psf
Mezzanine	250	No	20	None
Chiller Platforms	40	No	20	None
Existing Pickleball Roof	40	No	20	None
Existing Slab on Grade	300	No	20	None

[IBC 2021 table 1607.1]

Roof Load Criteria

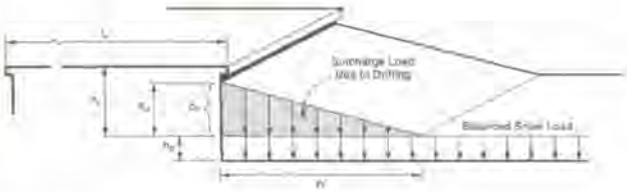
Roof Live Load =	20	psf	Existing Roof Load	[IBC 2021 table 1607.1]
Project Elevation (MSL) =	0	ft		
Ground Snow Load =	4	psf		[ASCE 7-22 figure 7.2-1]

Snow Load Values (From Enercalc)

Flat roof Snow Load, Pf = psf Balanced Snow Height, Hb = ft

Drift values:

Case	Drift height, hd	Drift Pd (psf)	W, ft



Wind Load Criteria

Exposure =	C		[ASCE 7-22 section 26.7.3]
Ultimate Design Wind Speed =	140	mph	[ASCE 7- Figure 26.5-1A - 26.5-1D]
Enclosure Classification:	Enclosed		[ASCE 7-22 Section 26.2]
GCPi (_ or -) =	0.18		[ASCE 7-22 Table 26.13-1]
K_d =	0.85		[ASCE 7-22 Table 26.6-1]
K_{zt} =	1.00		[ASCE 7-22 Section 26.8]
K_e =	1.00		[ASCE 7-22 Table 26.9-1]

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SHEET NO. 2 OF 2
PROJECT Auburndale Data Center
SUBJECT Design Criteria
CLIENT Nuclei JOB NO. 0
CALCULATED BY PD DATE 3/12/2026

Design Criteria - IBC 2021

Seismic Criteria (Including Supplement #3 of ASCE 7-22)

Lateral System(s):	Steel System Not Specifically Detailed for Seismic Resistance				[ASCE 7-22 Table 12.2-1]
R =	x-dir:	3.0	y-dir:	3.0	
Ω_0 =	x-dir:	3.0	y-dir:	3.0	
c_d =	x-dir:	3.0	y-dir:	3.0	
I_e =		1.00			[ASCE 7-22 Table 1.5-2]
Site Class :		D			
S_s :		0.075			https://ascehazardtool.org/
S_1 :		0.032			
Design Procedure:		Equivalent Lateral Force Procedure			
Ground Motion Hazard Requirement:		Not Required - Site Class D w/ Ss < 0.2			
Ground Motion Hazard Analysis Performed?		No	OK		
Site Response Analysis Performed?		No			
Section 11.4.8.1 Exception 1 applies?		Not Applicable			
Section 11.4.8.2 Exception 1 applies?		Not Applicable			
Section 20.3.1 Exception applies?		Not Applicable			
S_{M1} Increase Factor (11.4.8.1 Exception):		1.000			
S_{MS}:		0.1			[ASCE 7-22 Section 11.4-3]
S_{M1} :		0.1			[ASCE 7-22 Section 11.4-3]
S_{DS}:		0.067			[ASCE 7- Eqn 11.4-1]
S_{D1} :		0.046			[ASCE 7-22 Eqn 11.4-2]
Seismic Design category:		A			[ASCE 7-22 Section 11-6]
Are all Section 12.8.1.3 Criteria met?		No			[ASCE 7-22 Section 12.8.1.3]
Redundancy Factor, ρ =		1.0			[ASCE 7-22 Section 12.3.4]
T_L :		8.000			[ASCE 7-22 Table 22-14]
C_t :		0.020			[ASCE 7-22 Table 12.8-2]
x :		0.75			[ASCE 7-22 Table 12.8-2]
Cu =		1.4			[ASCE 7-22 Table 12.8-1]
h_n:		25.0	Mezzanine		[Structural height - section 11.2]
Tx (Calculated) =			(Leave Blank to use T = Ta)		[From RamSS Analysis]
Ty (Calculated) =			(Leave Blank to use T = Ta)		[From RamSS Analysis]
S_{DS} used in calculation of C_s		0.067			[ASCE 7-22 Section 12.8.1.3]
Ta =	0.224 sec				
Ts =	0.690 sec		1.5Ts =	1.035 sec	

Calculation of Cs:

Equation 12.8-2:	x-dir:	0.022	y-dir:	0.022	[ASCE 7-22 Eqn 12.8-2]
T-Used:	Tx =	0.224 sec	Ty =	0.224 sec	[ASCE 7-22 Section 12.8.2]
Eqn 12.8-3 C_{s_max} :	x-dir:	0.069	y-dir:	0.069	[ASCE 7-22 Eqn 12.8-4/ 12.8-5]
C_{s_min} :	x-dir:	0.010	y-dir:	0.010	[ASCE 7-22 Eqn 12.8-6/ 12.8-7]
Sec11.4.8.2 Cs =	x-dir:	N/A	y-dir:	N/A	
C_s Used:	x-dir:	0.022	y-dir:	0.022	

Foundation Criteria

System:	Shallow Spread Footings		
Allowable Bearing =	3000 psf	1/3 Increase Allowed?	
Coefficient of Friction =			
Active Fluid pressure =		psf/ft	
At Rest pressure =		psf/ft	
Passive Pressure =		psf/ft	
Seismic Fluid Pressure =		psf/ft	
Frost Depth =		inches	
Slab on grade modulus reaction =	100	psi/in	

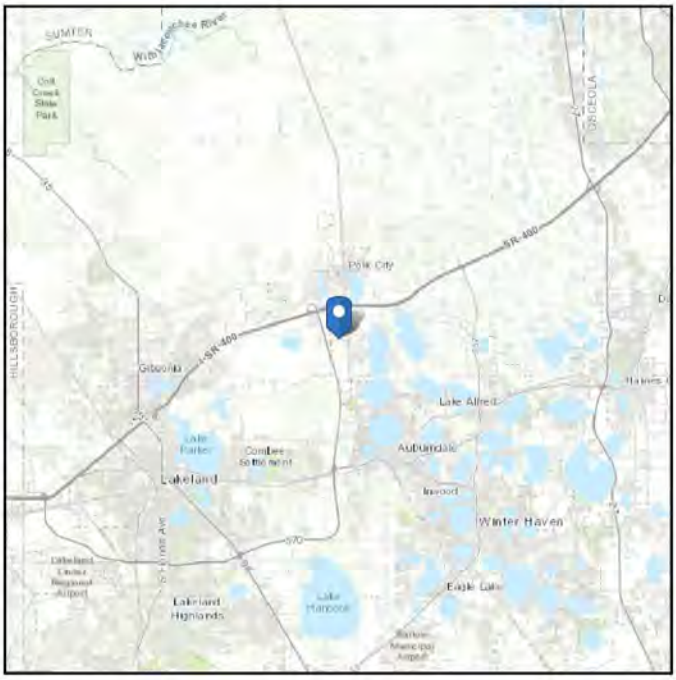
ASCE Hazards Report

Address:
810 Pace Rd
Auburndale, Florida
33823

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 28.136394
Longitude: -81.831448
Elevation: 0 ft (NAVD 88)





Wind

Results:

Wind Speed	140 Vmph
10-year MRI	80 Vmph
25-year MRI	93 Vmph
50-year MRI	105 Vmph
100-year MRI	114 Vmph
300-year MRI	124 Vmph
700-year MRI	140 Vmph
1,700-year MRI	147 Vmph
3,000-year MRI	150 Vmph
10,000-year MRI	160 Vmph
100,000-year MRI	170 Vmph
1,000,000-year MRI	180 Vmph

Drift limit but based on FBC 2023 Valt = Vult x square roof (0.6) = 108 mph Controls

Data Source: ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Fri May 01 2026

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-22 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years). Values for 10-year MRI, 25-year MRI, 50-year MRI and 100-year MRI are Service Level wind speeds, all other wind speeds are Ultimate wind speeds.

Site is in a hurricane-prone region as defined in ASCE/SEI 7-22 Section 26.2. Glazed openings shall be protected against wind-borne debris as specified in Section 26.12.3.

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ASCE Hazards Report

Address:
Pace Rd & Pearce Rd
Auburndale, Florida
33823

Standard:	ASCE/SEI 7-22	**Latitude:**	28.135728
Risk Category:	II	**Longitude:**	-81.830583
Soil Class:	D - Stiff Soil	**Elevation:**	176.3881404377274 ft (NAVD 88)





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Seismic

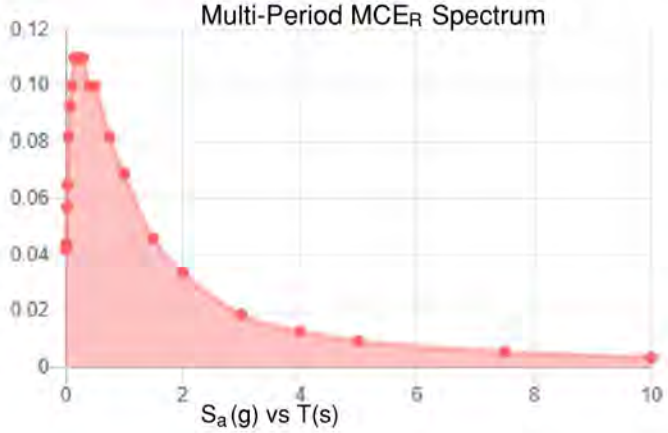
Site Soil Class: D - Stiff Soil

Results:

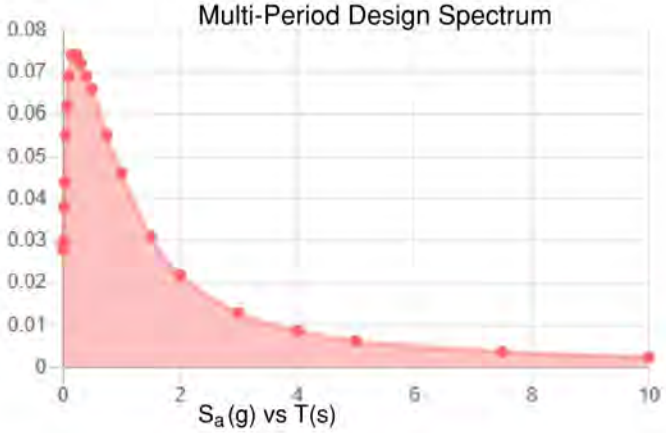
PGA_M :	0.037	T_L :	8
S_{MS} :	0.1	S_S :	0.075
S_{M1} :	0.069	S_1 :	0.032
S_{DS} :	0.067	V_{S30} :	260
S_{D1} :	0.046		

Seismic Design Category: A

Multi-Period MCE_R Spectrum



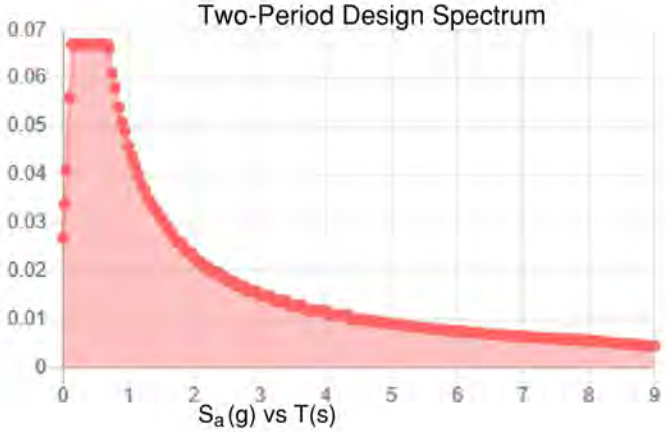
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: Thu Mar 12 2026

Date Source:
USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

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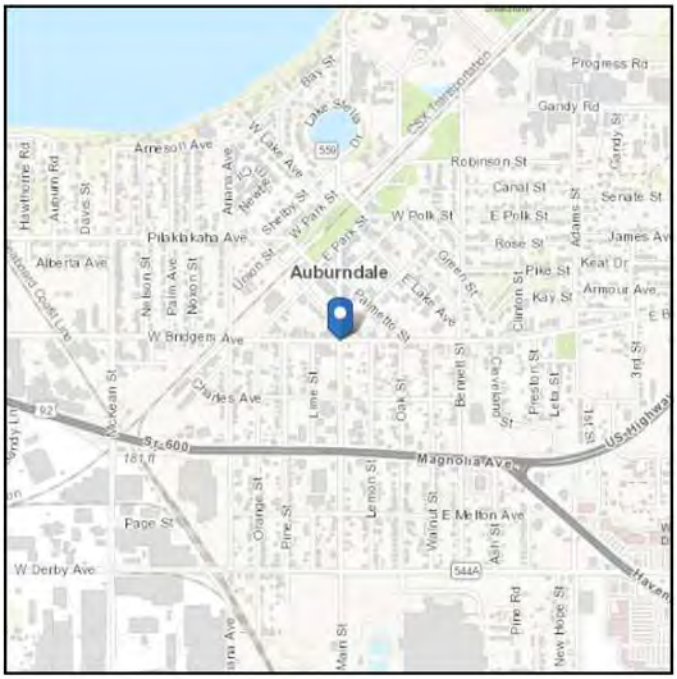
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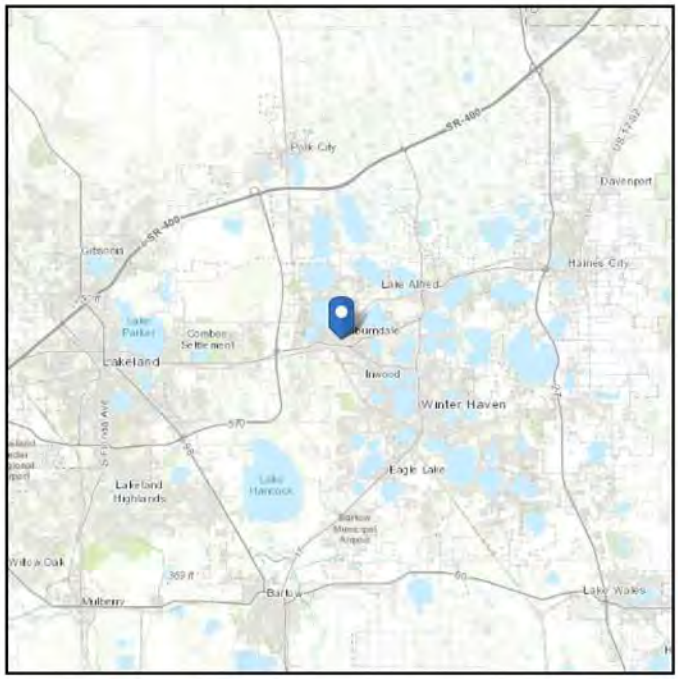
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ASCE Hazards Report

Address:
Auburndale
Florida,

Standard:	ASCE/SEI 7-22	**Latitude:**	28.062333
Risk Category:	II	**Longitude:**	-81.788689
Soil Class:	D - Stiff Soil	**Elevation:**	170.55071477848884 ft (NAVD 88)





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Snow

Results:

Ground Snow Load, p_g:	4 lb/ft^2
20-year MRI Value:	1 lb/ft^2
Winter Wind Parameter:	0.35
Mapped Elevation:	145.6 ft
Data Source:	ASCE/SEI 7-22, Figures 7.6-1 and 7.6-2 A-D
Date Accessed:	Tue Mar 24 2026

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

Ground Snow Loads for IRC only, $p_{g(asd)}$: 2.8 lb/ft^2

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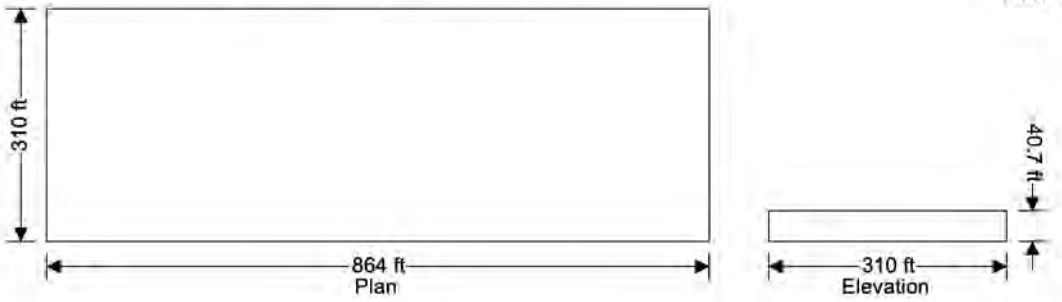
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COMPONENT AND CLADDING WIND LOADS PER ASCE7-22

In accordance with ASCE7-22

Using the components and cladding design method

Tedds calculation version 2.1.19



Building data

Type of roof	Flat
Length of building	b = **864.00** ft
Width of building	d = **310.00** ft
Height to eaves	H = **40.69** ft
Mean height	h = **40.69** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **16.28** ft

General wind load requirements

Basic wind speed	V = **140.0** mph
Risk category	II
Wind directionality factor (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.04**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_e \times V^2 \times 1psf/mph^2$ = **52.4** psf

Peak velocity pressure for internal pressure

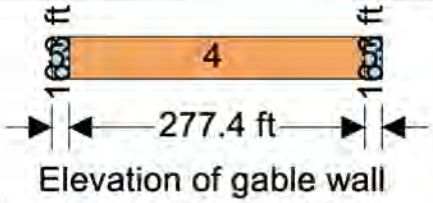
Peak velocity pressure – internal (as roof press.)	q_i = **52.36** psf

Equations used in tables

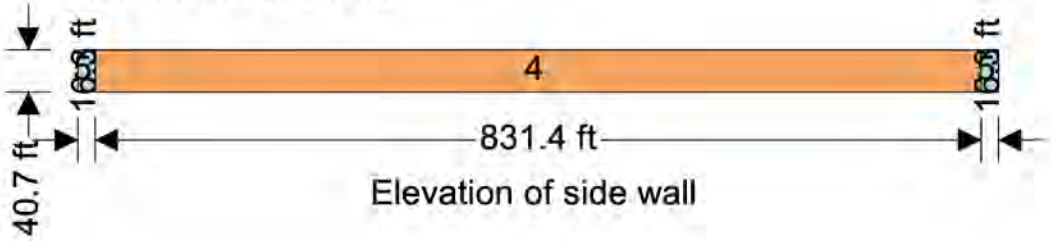
Net pressure	$p = K_d \times (q_h \times (GC_p - GC_{pi}))$

Components and cladding pressures - Wall (Table 30.3-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	48.1	-52.1
50 sf	4	-	-	50.0	0.79	-0.88	43.1	-47.1
200 sf	4	-	-	200.0	0.69	-0.78	38.9	-42.9
>500 sf	4	-	-	500.1	0.63	-0.72	36.0	-40.1
<=10 sf	5	-	-	10.0	0.90	-1.26	48.1	-64.1
50 sf	5	-	-	50.0	0.79	-1.04	43.1	-54.2
200 sf	5	-	-	200.0	0.69	-0.85	38.9	-45.7
>500 sf	5	-	-	500.1	0.63	-0.72	36.0	-40.1



Elevation of gable wall

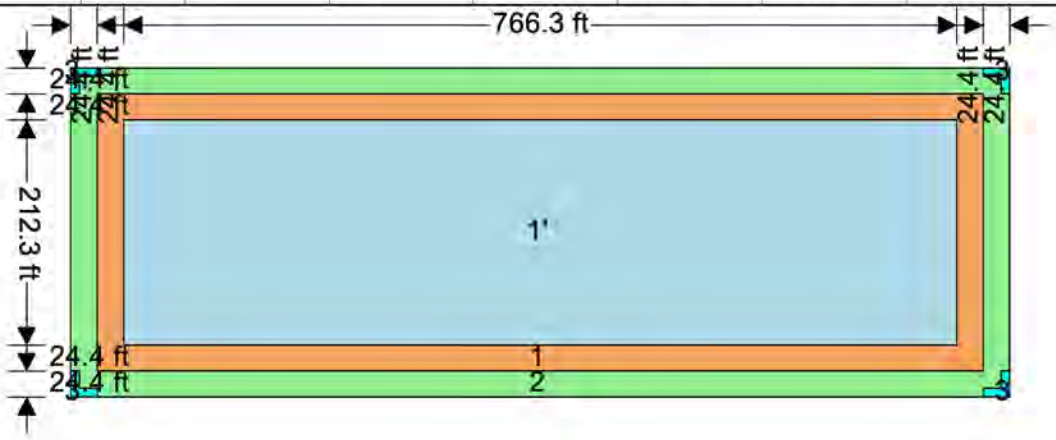


Elevation of side wall

Components and cladding pressures - Roof (Figure 30.3-2A)

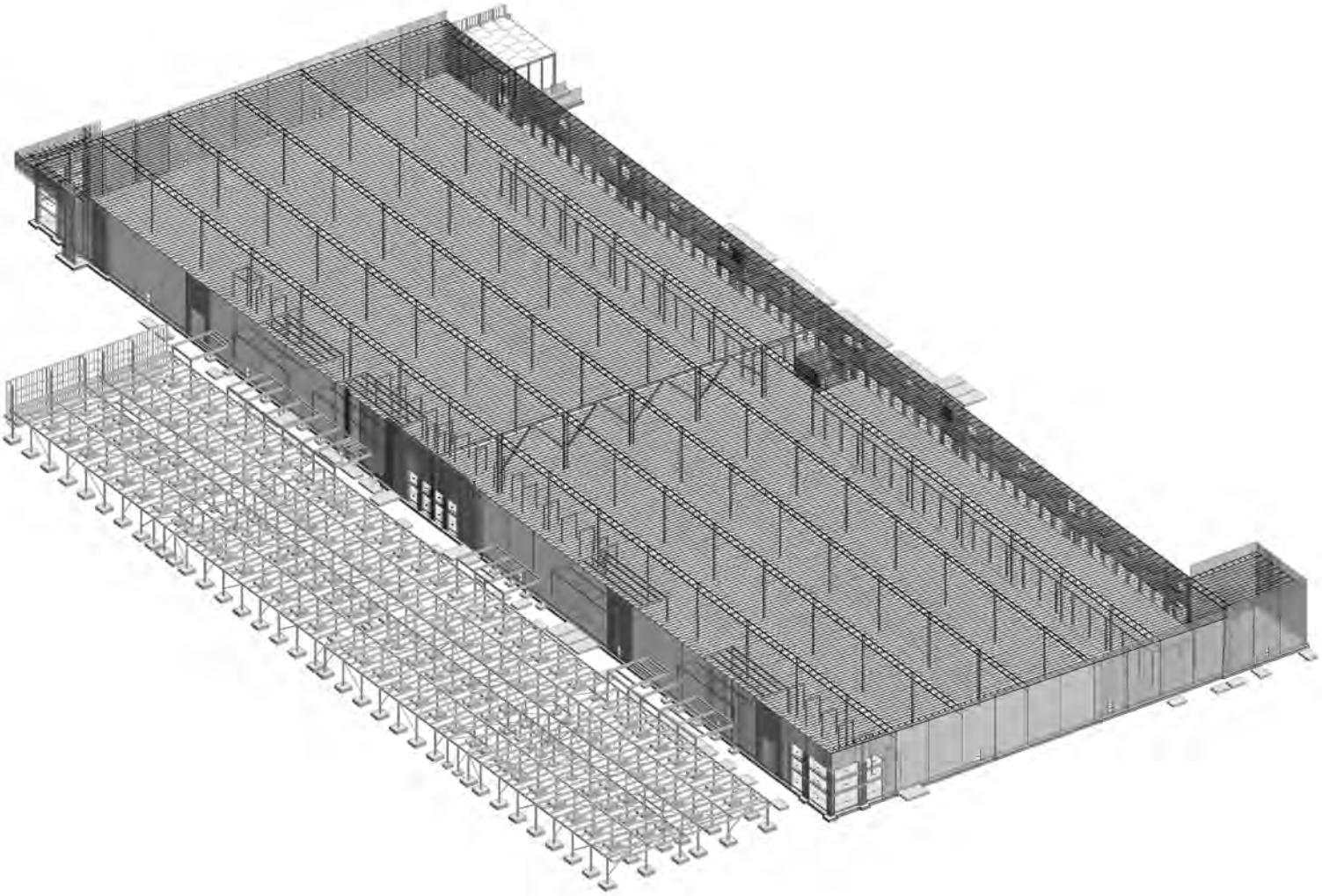
Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	21.4	-83.7
100 sf	1	-	-	100.0	0.20	-1.29	16.9	-65.3
200 sf	1	-	-	200.0	0.20	-1.16	16.9	-59.8
>500 sf	1	-	-	500.1	0.20	-1.00	16.9	-52.5
<=10 sf	1'	-	-	10.0	0.30	-0.90	21.4	-48.1
100 sf	1'	-	-	100.0	0.20	-0.90	16.9	-48.1
500 sf	1'	-	-	500.0	0.20	-0.55	16.9	-32.5
>1000 sf	1'	-	-	1000.1	0.20	-0.40	16.9	-25.8
<=10 sf	2	-	-	10.0	0.30	-2.30	21.4	-110.4
100 sf	2	-	-	100.0	0.20	-1.77	16.9	-86.8
200 sf	2	-	-	200.0	0.20	-1.61	16.9	-79.7
>500 sf	2	-	-	500.1	0.20	-1.40	16.9	-70.3
<=10 sf	3	-	-	10.0	0.30	-3.20	21.4	-150.4

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
100 sf	3	-	-	100.0	0.20	-2.14	16.9	-103.3
200 sf	3	-	-	200.0	0.20	-1.82	16.9	-89.1
>500 sf	3	-	-	500.1	0.20	-1.40	16.9	-70.3



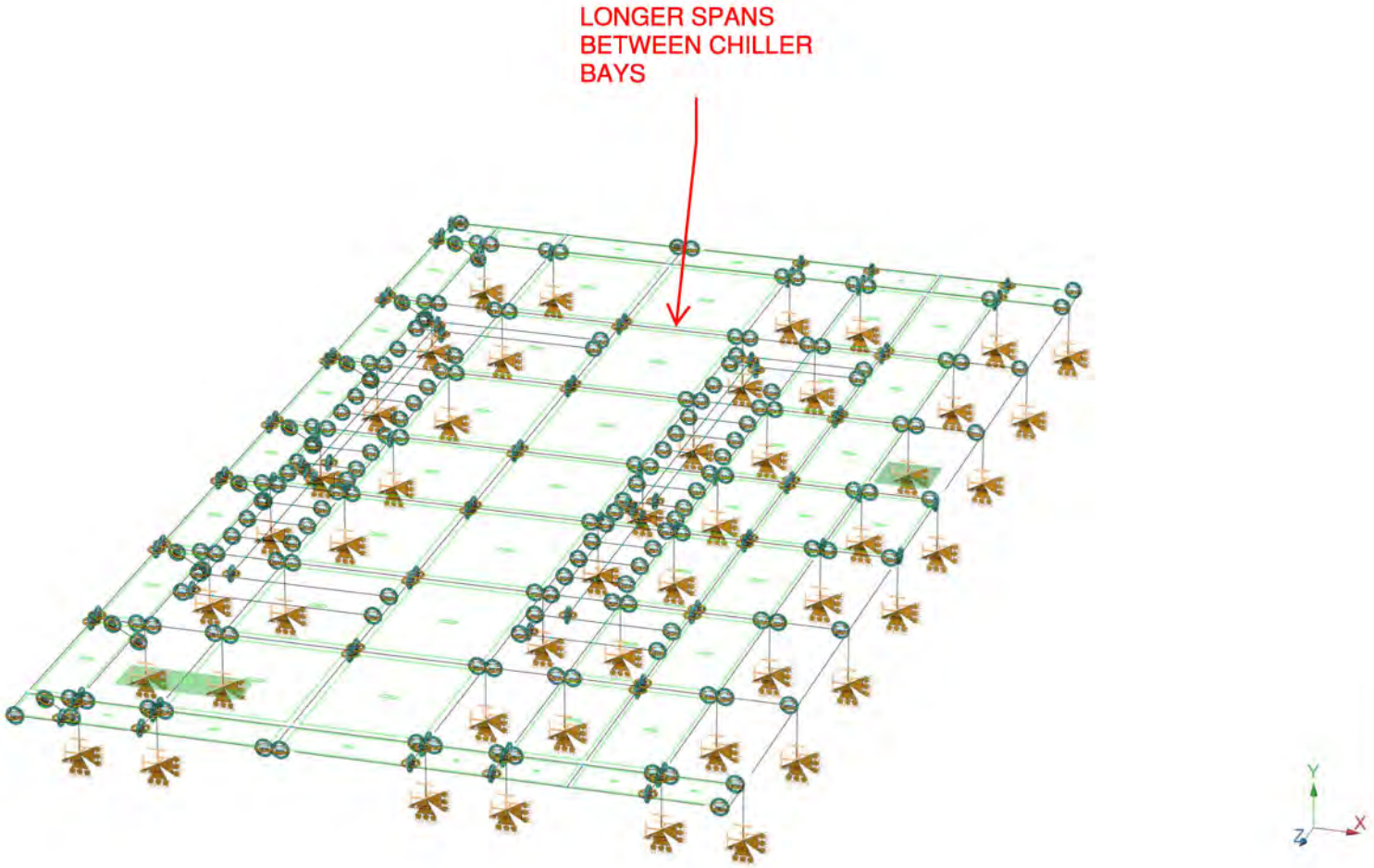
Plan on roof

Chapter 2 - Mechanical Equipment Platform with Acoustical Wall and Screen Wall



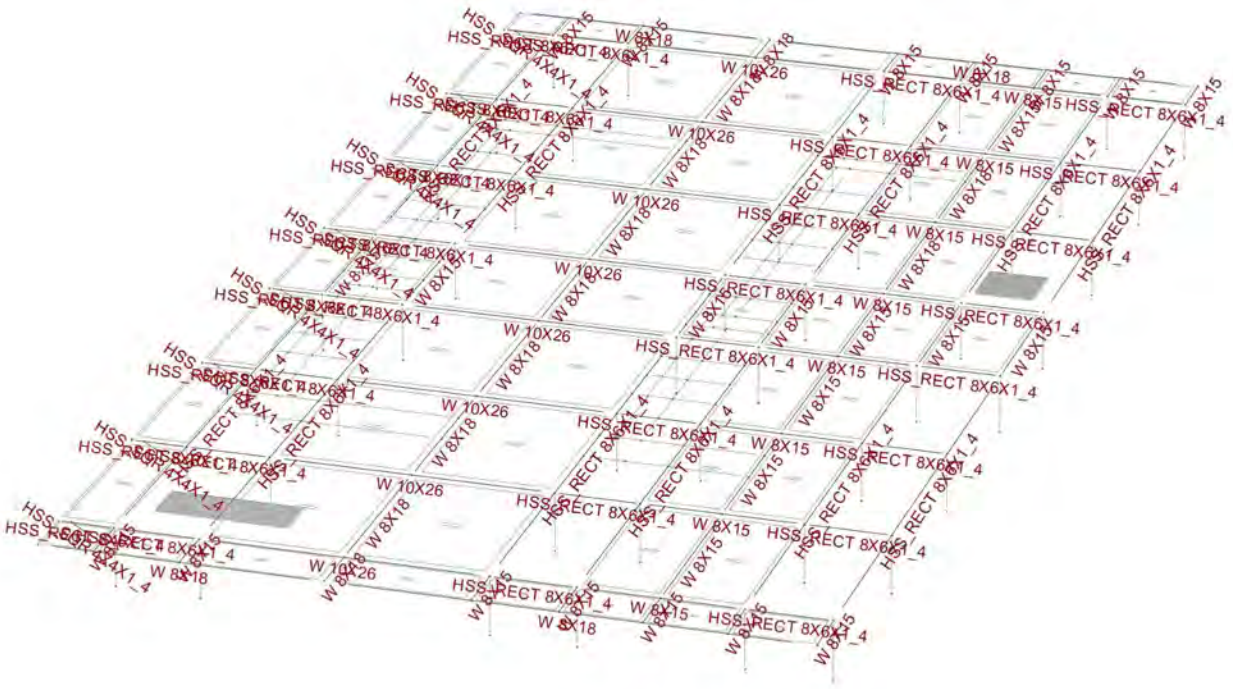
MECHANICAL EQUIPMENT PLATFORM

HINGES AND RESTRAINTS



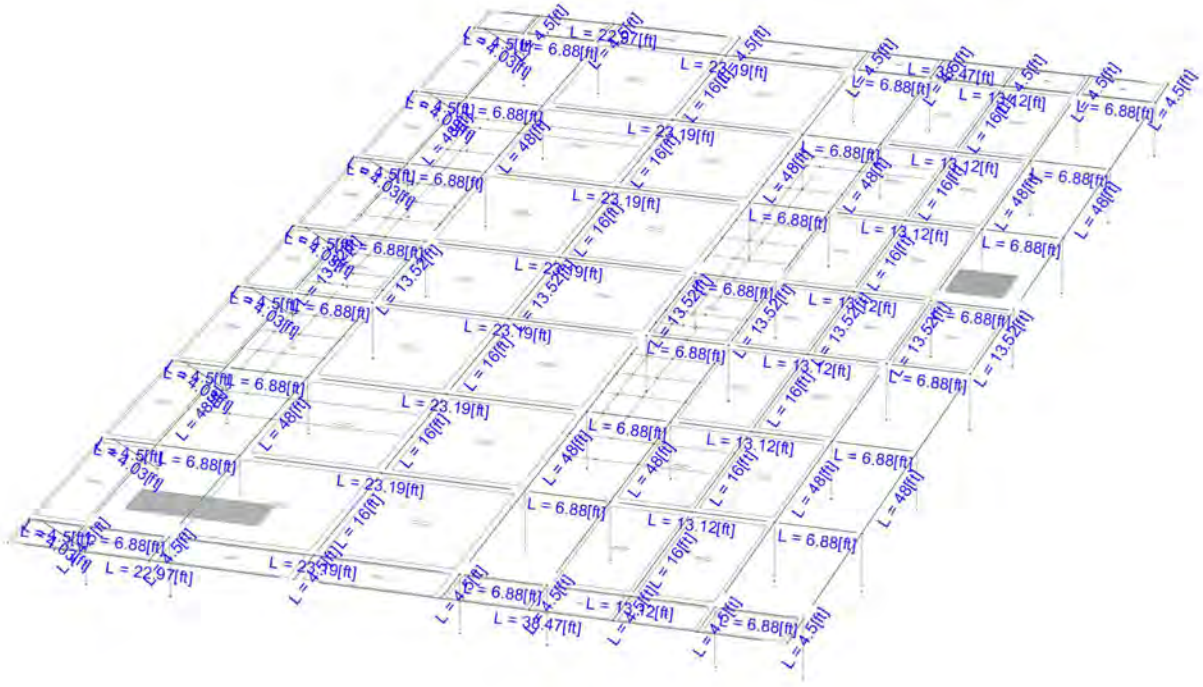
MECHANICAL EQUIPMENT PLATFORM

MEMBER SIZES



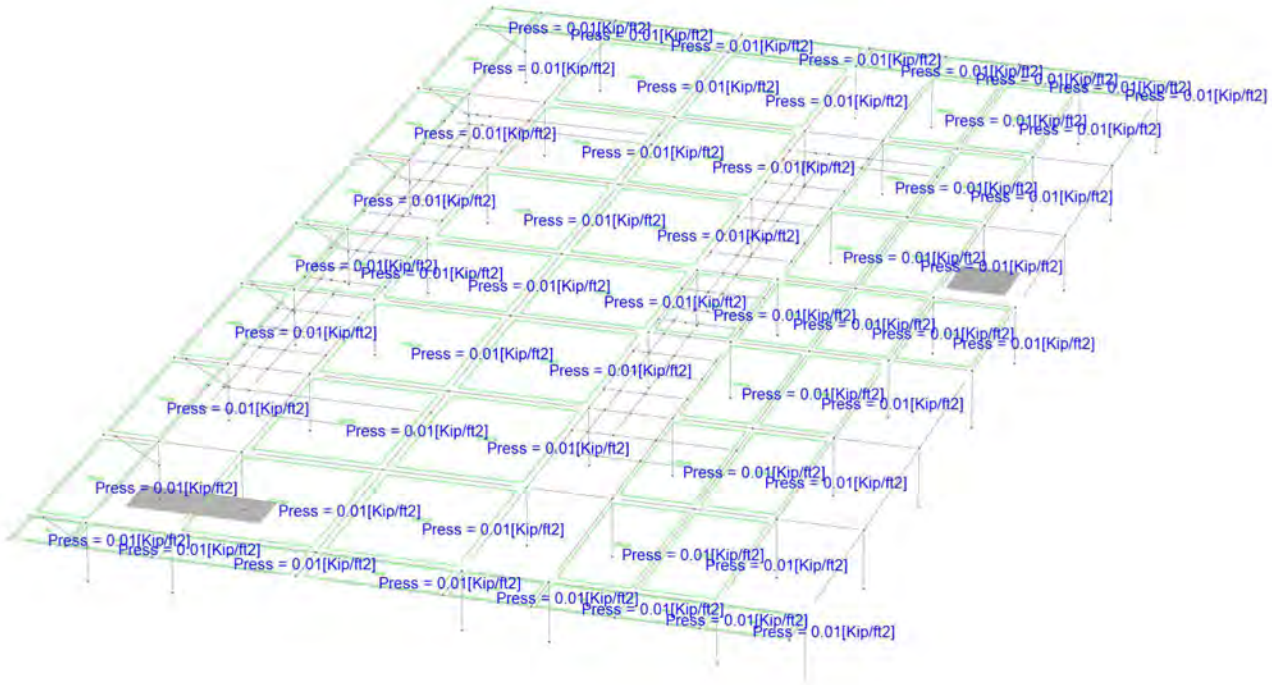
MECHANICAL EQUIPMENT PLATFORM

MEMBER LENGTHS



MECHANICAL EQUIPMENT PLATFORM

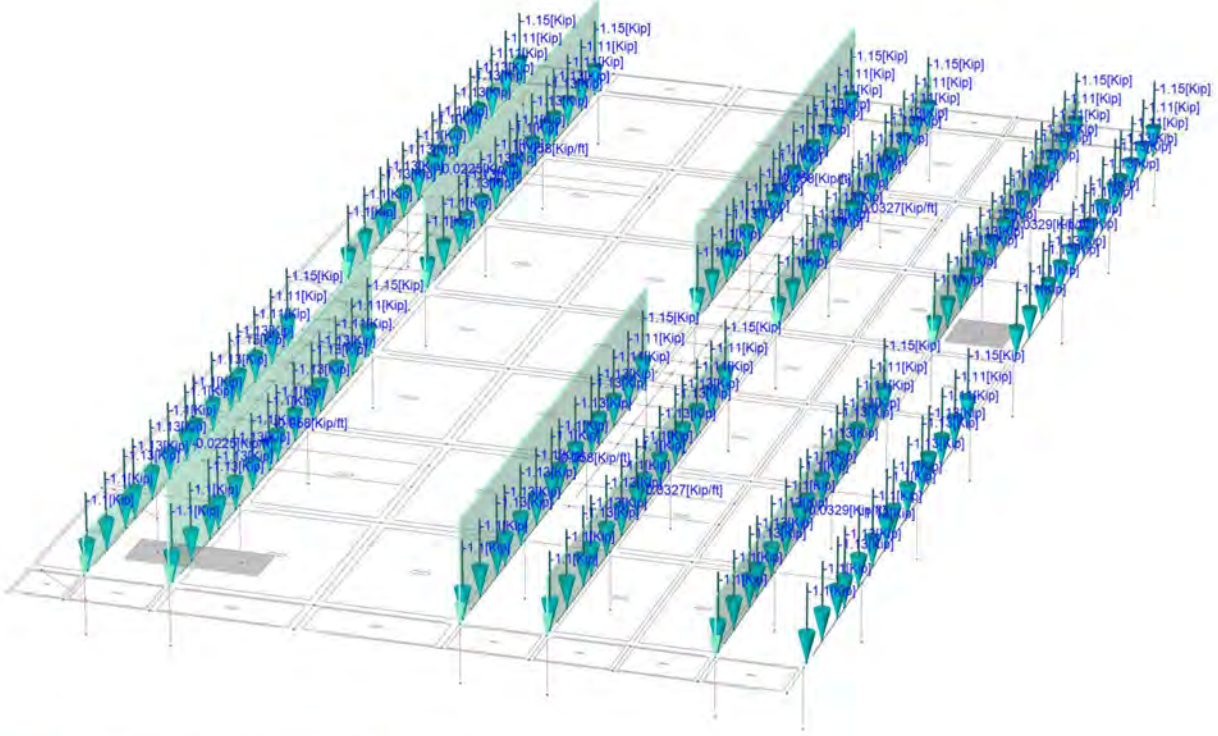
DEAD LOAD = 10 PSF



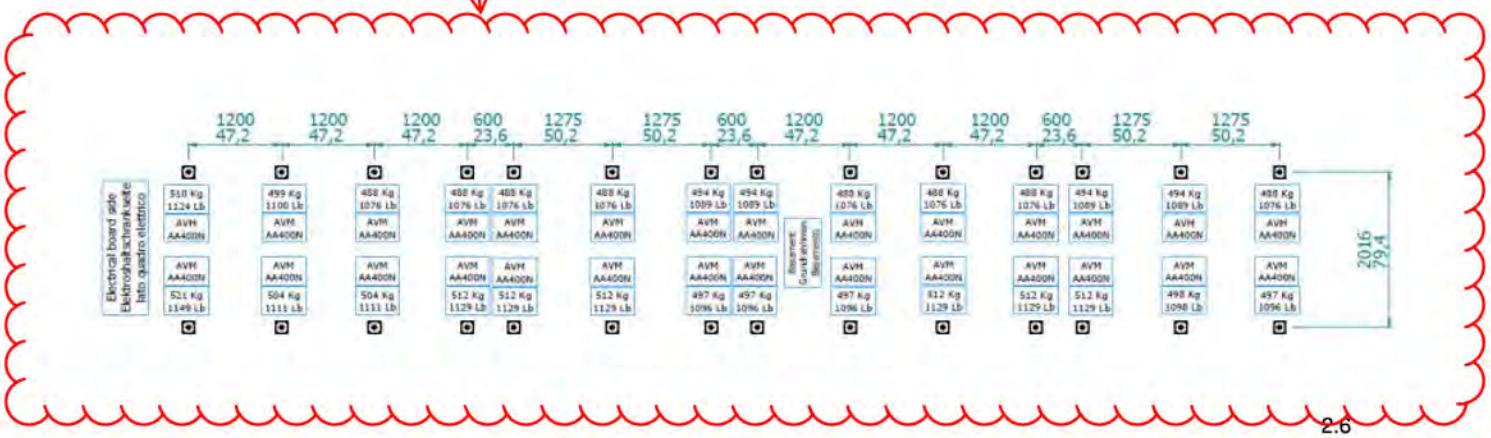
MECHANICAL EQUIPMENT PLATFORM

Loads
- Distributed area loads - Members
- Concentrated user loads - Members

DEAD LOAD = POINTS FOR EQUIPMENT



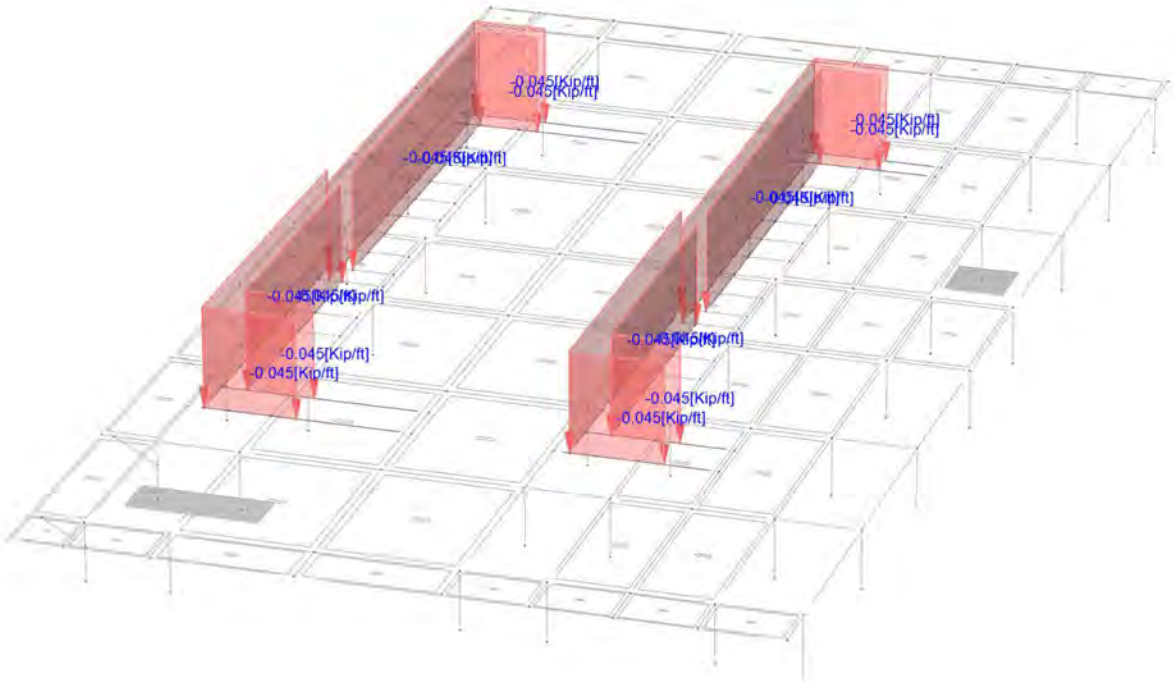
POINT LOADS FROM MANUF



MECHANICAL EQUIPMENT PLATFORM

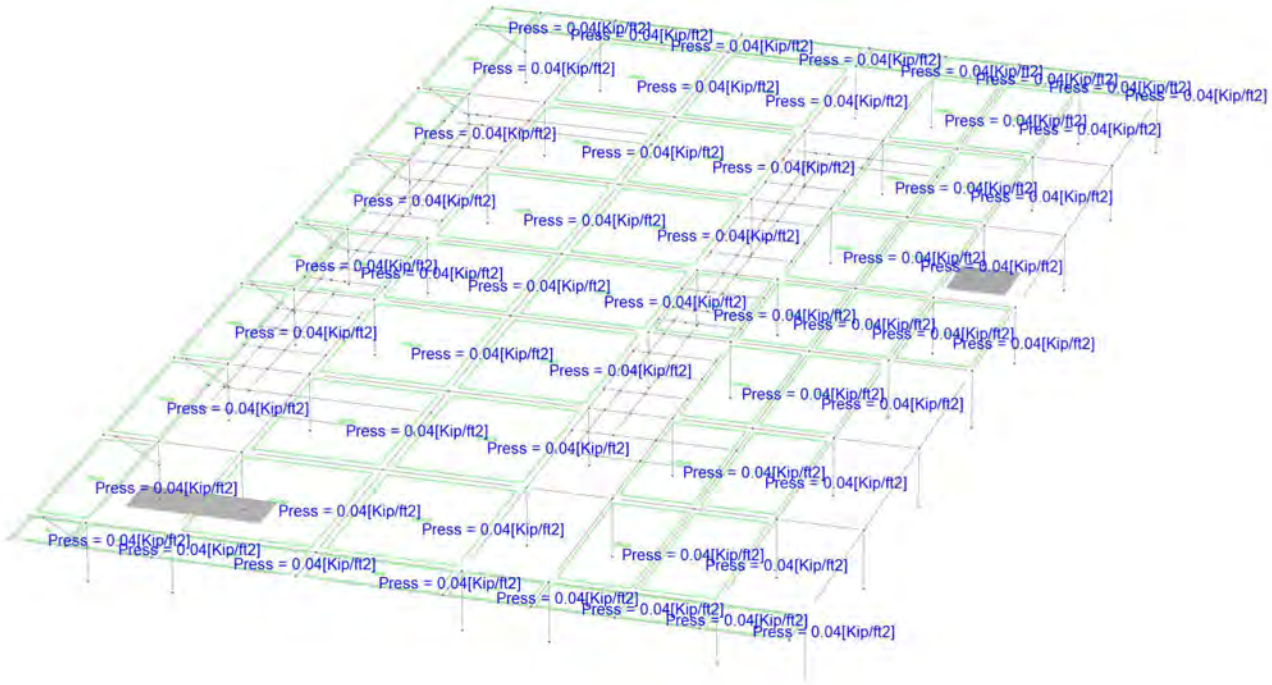
Loads
Distributed user loads - Members

DEAD LOAD 6-INCH PIPE = 45 PLF



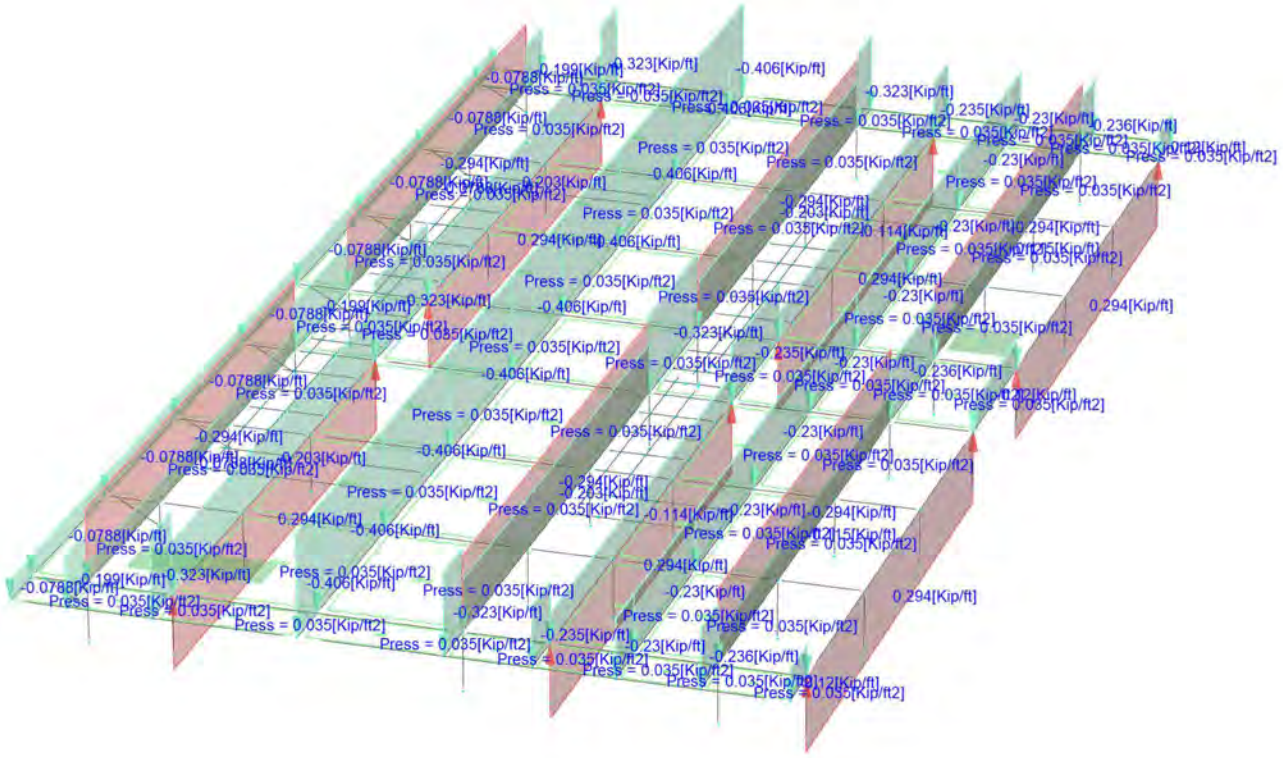
MECHANICAL EQUIPMENT PLATFORM

LIVE LOAD = 40 PSF



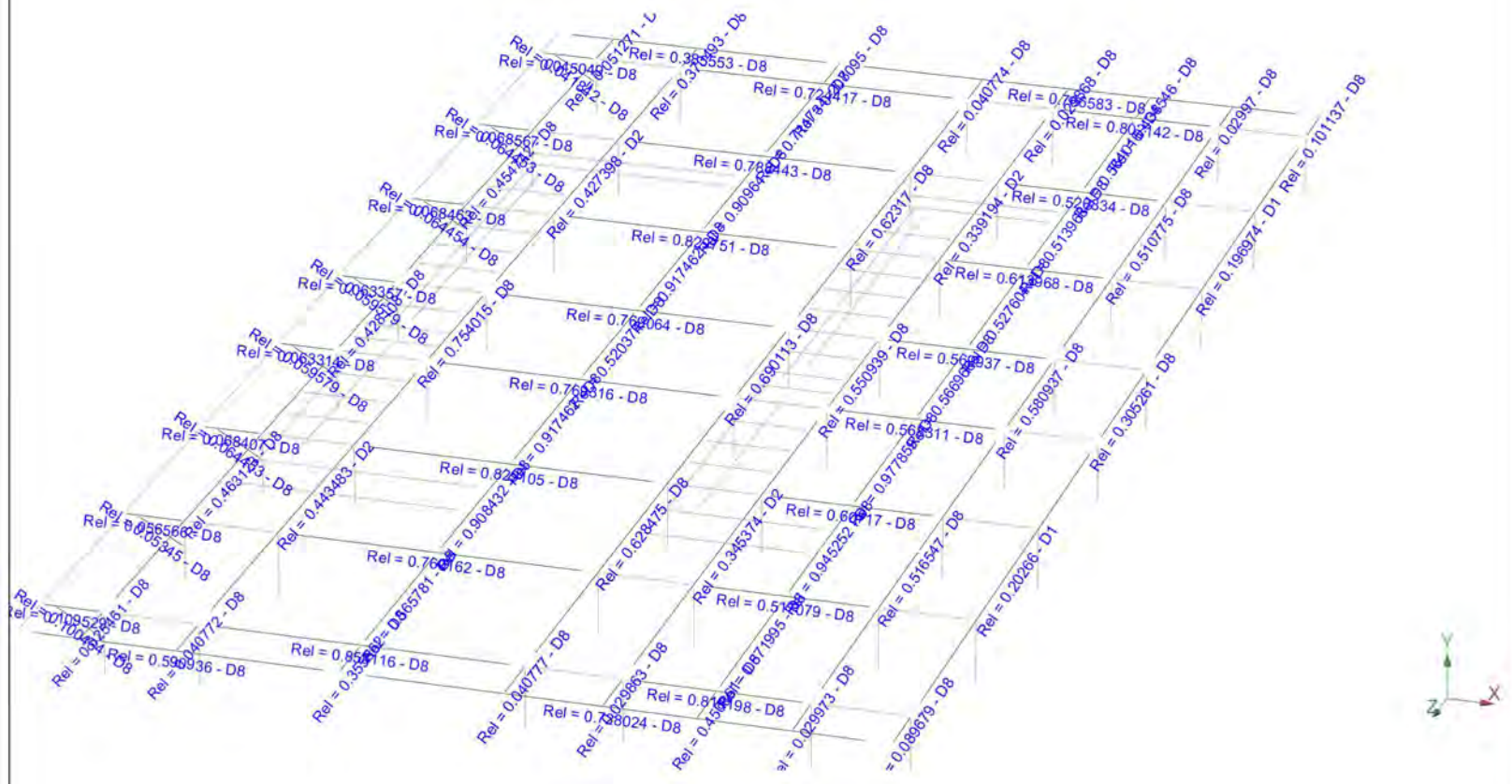
MECHANICAL EQUIPMENT PLATFORM

WIND LOADS



MECHANICAL EQUIPMENT PLATFORM

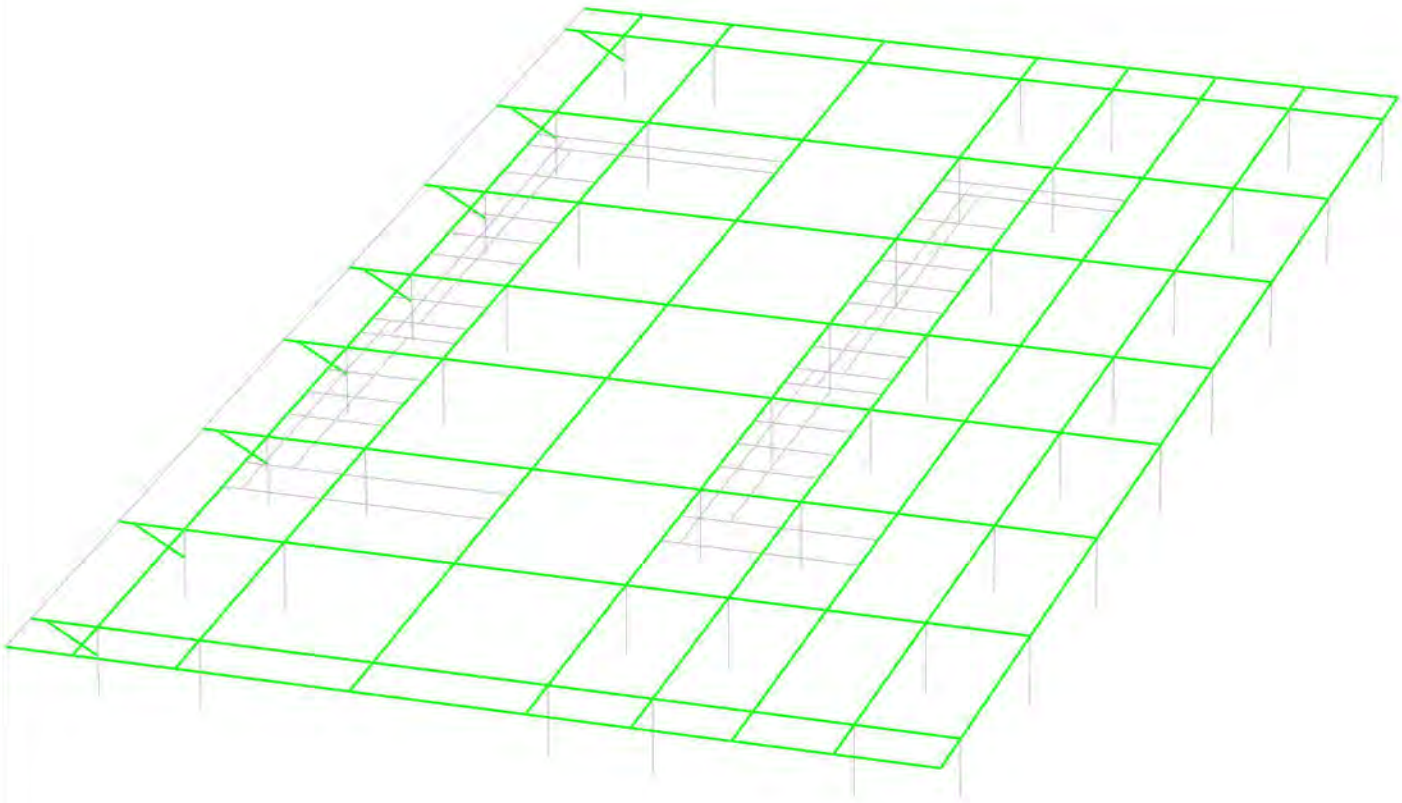
STRESS RATIO


Rel = 0.754015 - D8
Rel = 0.443483 - D2
Rel = 0.628475 - D8
Rel = 0.690113 - D8
Rel = 0.345374 - D2
Rel = 0.62317 - D8
Rel = 0.339194 - D2
Rel = 0.580937 - D8
Rel = 0.510775 - D8
Rel = 0.516547 - D8
Rel = 0.305261 - D8
Rel = 0.20266 - D1
Rel = 0.196974 - D1
Rel = 0.101137 - D8
Rel = 0.02997 - D8
Rel = 0.040774 - D8
Rel = 0.040772 - D8
Rel = 0.040777 - D8
Rel = 0.089679 - D8
Rel = 0.029973 - D8
Rel = 0.427398 - D2
Rel = 0.454133 - D8
Rel = 0.463193 - D8
Rel = 0.599936 - D8
Rel = 0.550939 - D8
Rel = 0.568311 - D8
Rel = 0.568937 - D8
Rel = 0.568668 - D8
Rel = 0.568583 - D8
Rel = 0.568342 - D8
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Rel = 0.568064 - D8
Rel = 0.568316 - D8
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Rel = 0.568162 - D8
Rel = 0.568079 - D8
Rel = 0.568198 - D8
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Rel = 0.568443 - D8
Rel = 0.568451 - D8
Rel = 0.568454 - D8
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Rel = 0.568461 - D8
Rel = 0.568407 - D8
Rel = 0.568314 - D8
Rel = 0.568357 - D8
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Rel = 0.568579 - D8
Rel = 0.568459 - D8
Rel = 0.568462 - D8
Rel = 0.568468 - D8
Rel = 0.568472 - D8
Rel = 0.568492 - D8
Rel = 0.568495 - D8
Rel = 0.568503 - D8
Rel = 0.568553 - D8
Rel = 0.568417 - D8
Rel = 0.568095 - D8
Rel = 0.568583 - D8
Rel = 0.568342 - D8
Rel = 0.568346 - D8
Rel = 0.568334 - D8
Rel = 0.568668 - D8
Rel = 0.568937 - D8
Rel = 0.568311

MECHANICAL EQUIPMENT PLATFORM

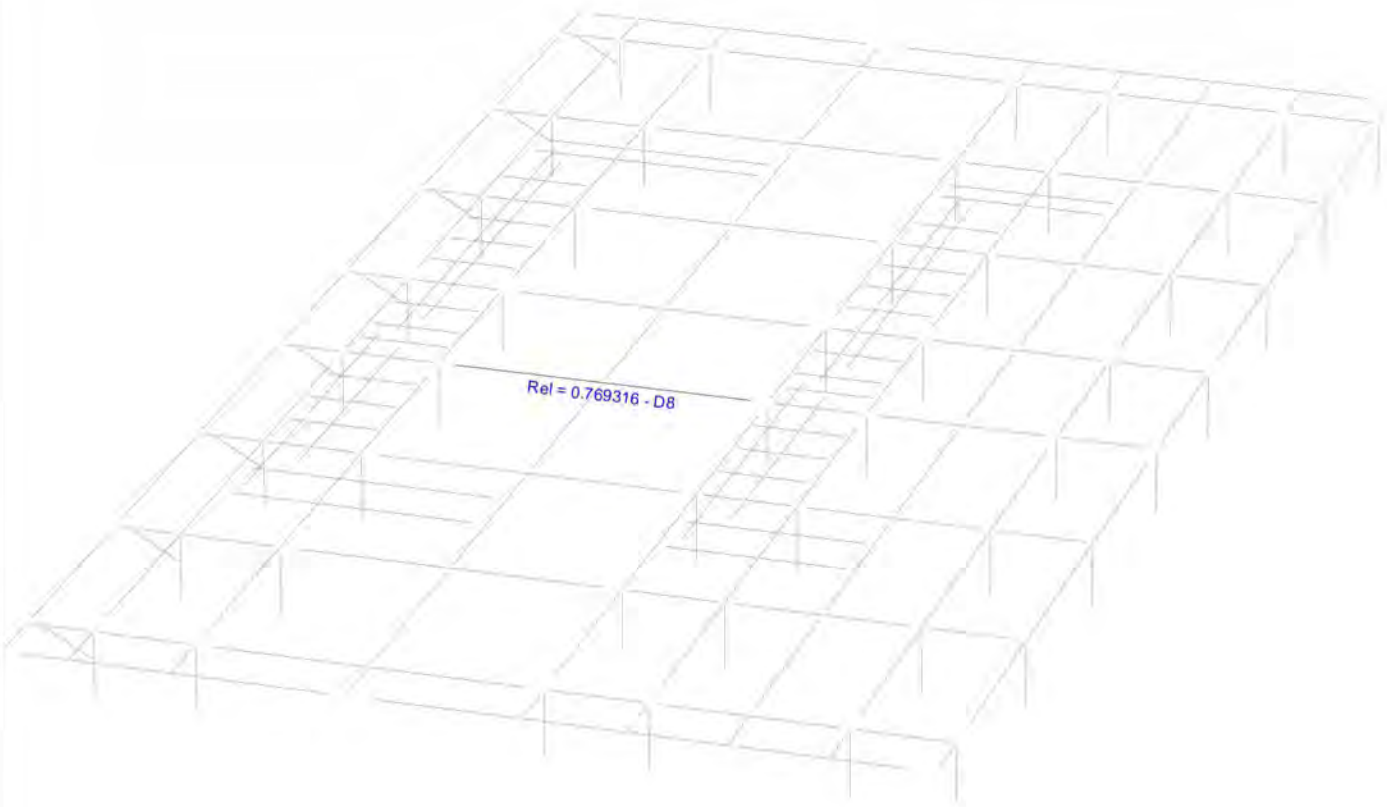
MEMBER STATUS

Design status
- Not designed
- Error on design
- Design O.K.
- With warnings



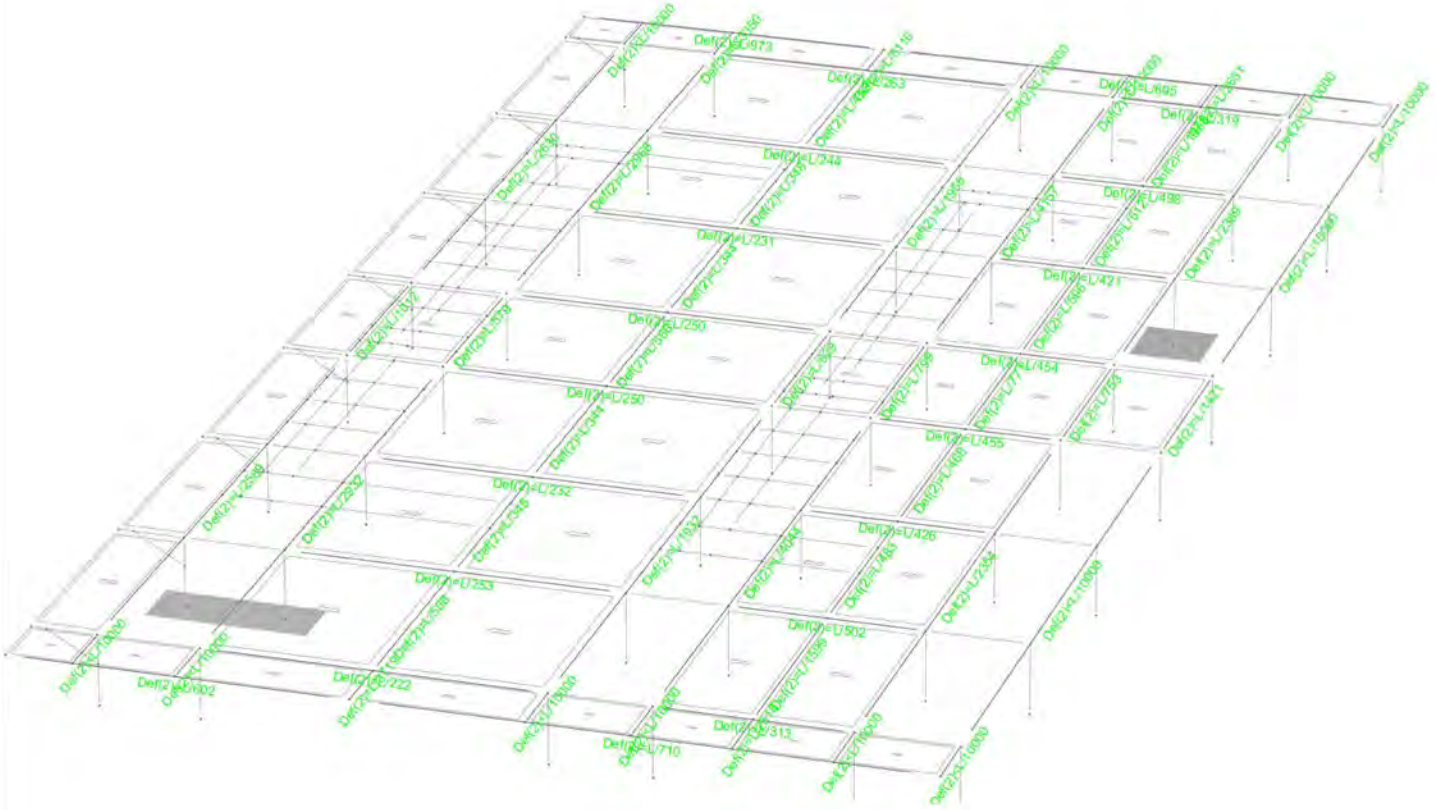
MECHANICAL EQUIPMENT PLATFORM

WF - CHECK FOR LONGER SPANS BETWEEN BAYS



MECHANICAL EQUIPMENT PLATFORM

DEFLECTION
DL + 0.75 LL + 0.45 WIND



RAM Elements Salas O'Brien

Current Date: 9/21/2026 1:15 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Chiller 1\2026-09-17 Chiller Equipment Platform Framing Plane - Between Platforms.retx

Steel Code Check Comprehensive Report

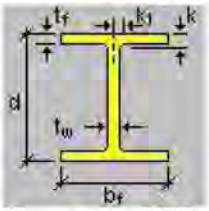
AISC 360-2016 LRFD (Hot-rolled)

Member : 133 (Walkway beam - long) - OK

Section information

Section name: W 10X26 **(US)**

Dimensions



bf	=	5.770	[in]	Width
d	=	10.300	[in]	Depth
k	=	0.740	[in]	Distance k
k1	=	0.688	[in]	Distance k1
tf	=	0.440	[in]	Flange thickness
tw	=	0.260	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.610	
Moment of Inertia (local axes) (I)	[in4]	144.000	14.100
Moment of Inertia (principal axes) (I')	[in4]	144.000	14.100
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.350	1.361
Radius of gyration (principal axes) (r')	[in]	4.350	1.361
Saint-Venant torsion constant. (J)	[in4]	0.402	
Section warping constant. (Cw)	[in6]	345.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	27.900	4.890
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	27.900	4.890
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	27.900	4.890
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	27.900	4.890
Plastic section modulus (local axis) (Z)	[in3]	31.300	7.500
Plastic section modulus (principal axis) (Z')	[in3]	31.300	7.500
Polar radius of gyration. (ro)	[in]	4.558	
Area for shear (Aw)	[in2]	5.080	2.680
Torsional constant. (C)	[in3]	0.876	

Material : A992 Gr50

WF - CHECK FOR LONGER SPANS BETWEEN BAYS

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	23.19

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.59	11.59
11.59	11.59

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
23.19	23.19	23.19	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.00			
Capacity	:	342.45 [Kip]	Reference	:	Cl.D2
Demand	:	1.33 [Kip]	Ctrl Eq.	:	D8 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	342.45	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	380.50	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

WF - CHECK FOR LONGER SPANS BETWEEN BAYS

Ratio	:	0.00			
Capacity	:	253.92 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.56	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	33.92	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	253.92	Cl.E3
Effective length (KL_{33})	[ft]	23.19	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	63.96	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	69.97	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.61	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	37.07	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	282.13	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	41.15 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.56	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	33.92	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	41.15	Cl.E3
Effective length (KL_{22})	[ft]	23.19	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	204.40	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	6.85	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.61	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	6.01	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	45.72	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	192.80	Cl.E4
Effective length (KL_{11})	[ft]	23.19	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	36.43	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	7.61	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	28.15	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	214.22	Eq.E4-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.77			
Capacity	:	117.38 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	90.00 [Kip*ft]	Ctrl Eq.	:	D8 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.56	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	33.92	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	117.38	Cl.F2.1
Yielding (M_n)	[Kip*ft]	130.42	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	117.38	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	4.81	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.58	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	14.94	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.66	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	130.42	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	28.13 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.02 [Kip*ft]	Ctrl Eq.	:	D8 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.56	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	33.92	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	28.13	Cl.F6.1
Yielding (M_n)	[Kip*ft]	31.25	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	137.10 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D8 at 50.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	137.10	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	152.33	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.10			
Capacity	:	80.40 [Kip]	Reference	:	Cl.G1
Demand	:	7.96 [Kip]	Ctrl Eq.	:	D8 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	80.40	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	80.40	Eq.G2-1

Combined Actions Design ✔

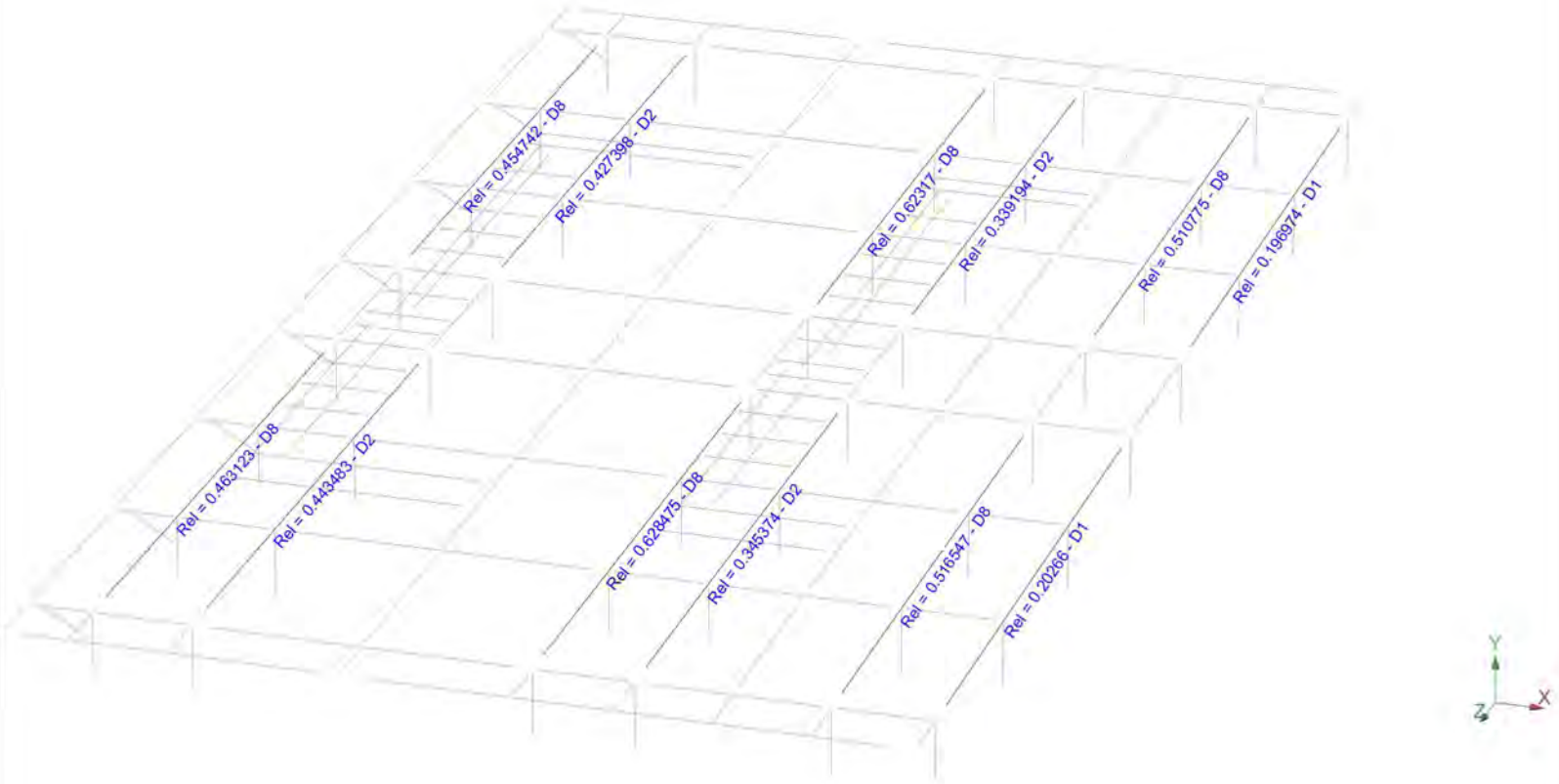
Combined flexure and axial

Ratio	:	0.77			
Ctrl Eq.	:	D8 at 50.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.77	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	117.38	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	28.13	Cl.H1.1
Available axial strength (P_c)	[Kip]	342.45	Cl.H1.1

MECHANICAL EQUIPMENT PLATFORM

HSS MEMBER CHECK FOR EQUIPMENT



RAM Elements Salas O'Brien

Current Date: 9/21/2026 1:14 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Chiller 1\2026-09-17 Chiller Equipment Platform Framing Plane - Between Platforms.retx

Steel Code Check Comprehensive Report

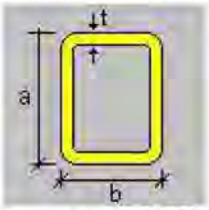
AISC 360-2016 LRFD (Hot-rolled)

Member : 95 (Platform Beam Long) - OK

Section information

Section name: HSS_RECT 8X6X1_4 **(US)**

Dimensions



a	=	8.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	6.170	
Moment of Inertia (local axes) (I)	[in4]	56.600	36.400
Moment of Inertia (principal axes) (I')	[in4]	56.600	36.400
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.029	2.429
Radius of gyration (principal axes) (r')	[in]	3.029	2.429
Saint-Venant torsion constant. (J)	[in4]	70.300	
Section warping constant. (Cw)	[in6]	5.873	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.200	12.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.200	12.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.200	12.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.200	12.100
Plastic section modulus (local axis) (Z)	[in3]	16.900	13.900
Plastic section modulus (principal axis) (Z')	[in3]	16.900	13.900
Polar radius of gyration. (ro)	[in]	3.883	
Area for shear (Aw)	[in2]	2.470	3.402
Torsional constant. (C)	[in3]	20.823	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	48.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
48.00	48.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
48.00	48.00	48.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	277.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.27 [Kip]	Ctrl Eq.	:	D2 at 51.39%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	277.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	308.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	38.54 [Kip]	Reference	:	Cl.E3
Demand	:	0.51 [Kip]	Ctrl Eq.	:	D2 at 66.67%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	22.75	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.33	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	38.54	Cl.E3
Effective length (KL_{33})	[ft]	48.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	190.18	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	7.91	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.17	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	6.94	Eq.E3-3
Nominal flexural buckling strength (P_{n33})	[Kip]	42.82	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.02			
Capacity	:	24.79 [Kip]	Reference	:	Cl.E3
Demand	:	0.51 [Kip]	Ctrl Eq.	:	D2 at 66.67%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	22.75	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.33	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	24.79	Cl.E3
Effective length (KL_{22})	[ft]	48.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	237.15	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	5.09	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.17	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	4.46	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	27.54	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.33			
Capacity	:	63.38 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-21.21 [Kip*ft]	Ctrl Eq.	:	D2 at 66.67%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	22.75	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	31.33	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	63.38	Cl.F7.1
Yielding (M_n)	[Kip*ft]	70.42	Eq.F7-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	63.38	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	18.81	Eq.F7-12
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	491.95	Eq.F7-13
Lateral-torsional buckling modification factor (C_b)	--	2.81	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	70.42	Eq.F7-10

Bending about minor axis, M22

Ratio	:	0.01		
Capacity	:	47.77 [Kip*ft]	Reference	: Cl.F7.2
Demand	:	-0.57 [Kip*ft]	Ctrl Eq.	: D8 at 33.33%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	22.75	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	31.33	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	52.13	Cl.F7.1
Yielding (M_n)	[Kip*ft]	57.92	Eq.F7-1
Factored web local buckling strength about a geometric axis(ϕM_n):	[Kip*ft]	47.77	Cl.F7.2
Local web buckling (M_n)	[Kip*ft]	53.08	Eq.F7-2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	66.70 [Kip]	Reference	: Cl.G1
Demand	:	0.18 [Kip]	Ctrl Eq.	: D8 at 25.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	66.70	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	74.11	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.08		
Capacity	:	91.86 [Kip]	Reference	: Cl.G1
Demand	:	6.97 [Kip]	Ctrl Eq.	: D2 at 66.67%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	91.86	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	102.07	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.02			
Capacity	:	46.85 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.76 [Kip*ft]	Ctrl Eq.	:	D8 at 33.33%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	46.85	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	52.06	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.35			
Ctrl Eq.	:	D2 at 66.67%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.35	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	63.38	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	47.77	Cl.H1.1
Available axial strength (P_c)	[Kip]	24.79	Cl.H1.1

RAM Elements Salas O'Brien

Current Date: 9/21/2026 4:34 PM
Units system: English

Design Results
Reinforced Concrete Footings

General Information

Global status	:	OK
Design Code	:	ACI 318-2019
Footing type	:	Combined
Column type	:	Pedestal

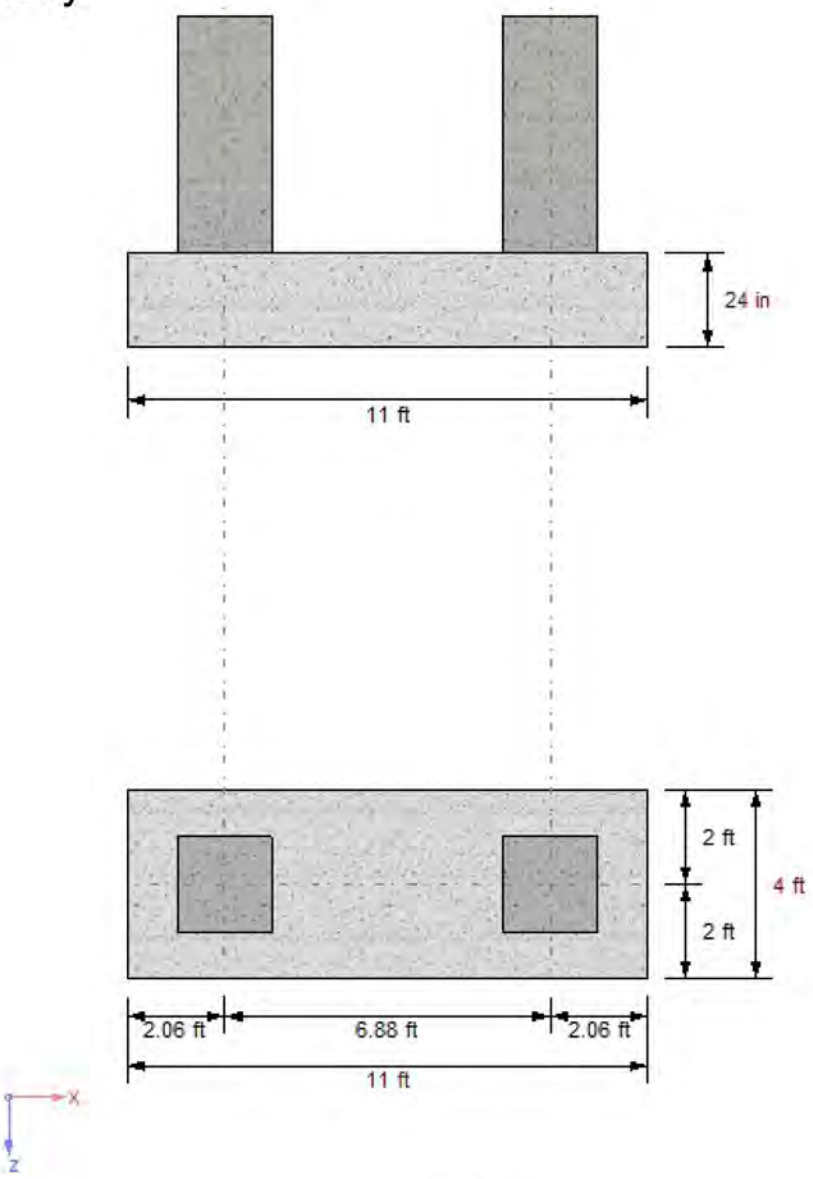
Materials

Concrete, f'c	:	3.00 [Kip/in2]	Steel, fy	:	60.00 [Kip/in2]
Concrete type	:	Normal	Epoxy coated	:	No
Concrete elasticity modulus	:	3122.02 [Kip/in2]	Steel elasticity modulus	:	29000.00 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]			

Soil

Modulus of subgrade reaction	:	200.00 [Kip/ft3]
Unit weight (wet)	:	0.11 [Kip/ft3]

Geometry



Length	:	11.00 [ft]
Width	:	4.00 [ft]
Thickness	:	2.00 [ft]
Base depth	:	4.00 [ft]
Base area	:	44.00 [ft2]
Footing volume	:	88.00 [ft3]
Pedestal length 1	:	24.00 [in]
Pedestal width 1	:	24.00 [in]
Pedestal height 1	:	60.00 [in]
Pedestal length 2	:	24.00 [in]
Pedestal width 2	:	24.00 [in]
Pedestal height 2	:	60.00 [in]
Distance between pedestals	:	6.88 [ft]
Pedestal location relative to footing g.c.	:	Centered

Reinforcement

Longitudinal reinforcement

Free cover	:	3.00 [in]
Maximum Rho/Rho balanced ratio	:	0.75
Bottom reinforcement // to L (xx)	:	5-#6 @ 10.00"
Top reinforcement // to L (xx)	:	5-#6 @ 10.00"
Bottom reinforcement // to B (zz)	:	6-#6 @ 9.00" (Zone 1)
Bottom reinforcement // to B (zz)	:	3-#6 @ 13.00" (Zone 2)
Bottom reinforcement // to B (zz)	:	6-#6 @ 9.00" (Zone 3)

Pedestal reinforcement 1

Longitudinal	:	12 - #7
Free cover	:	1.00 [in]
Provided area	:	7.20 [in2]
Bars number // to x axis	:	4
Bars number // to z axis	:	4
Transverse	:	#6 @ 14.00"
Legs number // to x axis	:	3
Legs number // to z axis	:	3

Pedestal reinforcement 2

Longitudinal	:	12 - #7
Free cover	:	1.00 [in]
Provided area	:	7.20 [in2]
Bars number // to x axis	:	4
Bars number // to z axis	:	4
Transverse	:	#6 @ 14.00"
Legs number // to x axis	:	3
Legs number // to z axis	:	3

Load Conditions

Service loads:

S1	:	_S1
S2	:	_S2
S3	:	_S3
S4	:	_S4
S5	:	_S5
S6	:	_S6
S7	:	_S7
S8	:	_S8
S9	:	_S9

Design strength loads:

D1	:	_D1
D2	:	_D2
D3	:	_D3
D4	:	_D4
D5	:	_D5
D6	:	_D6
D7	:	_D7
D8	:	_D8
D9	:	_D9
D10	:	_D10

Loads

Condition	Footing	Node	Column	Axial [Kip]	Mxx [Kip*ft]	Mzz [Kip*ft]	Vx [Kip]	Vz [Kip]
DL	1	144	1	8.97	0.73	-0.37	0.43	0.55
LL	1	144	1	2.82	0.36	-0.69	0.96	0.11
Wx	1	144	1	0.00	0.00	0.00	0.00	0.00
Wz	1	144	1	7.75	0.83	-0.62	0.84	0.58
_D1	1	144	1	12.56	1.02	-0.51	0.60	0.76
_D2	1	144	1	15.27	1.45	-1.54	2.06	0.83
_D3	1	144	1	10.76	0.87	-0.44	0.51	0.65
_D4	1	144	1	14.64	1.29	-0.75	0.93	0.94
_D5	1	144	1	10.76	0.87	-0.44	0.51	0.65
_D6	1	144	1	18.51	1.70	-1.06	1.35	1.23
_D7	1	144	1	13.58	1.23	-1.13	1.48	0.76
_D8	1	144	1	21.33	2.06	-1.75	2.32	1.34
_D9	1	144	1	8.07	0.65	-0.33	0.39	0.49
_D10	1	144	1	15.82	1.48	-0.95	1.22	1.07
_S1	1	144	1	8.97	0.73	-0.37	0.43	0.55
_S2	1	144	1	11.78	1.09	-1.05	1.39	0.66
_S3	1	144	1	11.08	1.00	-0.88	1.15	0.63
_S4	1	144	1	8.97	0.73	-0.37	0.43	0.55
_S5	1	144	1	13.62	1.22	-0.74	0.93	0.89
_S6	1	144	1	11.08	1.00	-0.88	1.15	0.63
_S7	1	144	1	14.57	1.37	-1.16	1.53	0.89
_S8	1	144	1	5.38	0.44	-0.22	0.26	0.33
_S9	1	144	1	10.03	0.93	-0.59	0.76	0.67
DL	1	145	2	10.34	0.78	-0.95	-0.19	0.50
LL	1	145	2	7.61	0.31	-2.08	-0.47	0.39
Wx	1	145	2	0.00	0.00	0.00	0.00	0.00
Wz	1	145	2	1.37	-0.26	-1.82	-0.41	-0.14
_D1	1	145	2	14.48	1.10	-1.33	-0.26	0.70
_D2	1	145	2	24.58	1.44	-4.47	-0.98	1.22
_D3	1	145	2	12.41	0.94	-1.14	-0.22	0.60
_D4	1	145	2	13.10	0.81	-2.06	-0.43	0.53
_D5	1	145	2	12.41	0.94	-1.14	-0.22	0.60
_D6	1	145	2	13.78	0.68	-2.97	-0.63	0.47
_D7	1	145	2	20.02	1.26	-3.22	-0.70	0.99
_D8	1	145	2	21.38	0.99	-5.05	-1.10	0.85
_D9	1	145	2	9.31	0.71	-0.86	-0.17	0.45
_D10	1	145	2	10.68	0.44	-2.68	-0.57	0.31
_S1	1	145	2	10.34	0.78	-0.95	-0.19	0.50
_S2	1	145	2	17.95	1.10	-3.03	-0.66	0.89
_S3	1	145	2	16.05	1.02	-2.51	-0.54	0.79
_S4	1	145	2	10.34	0.78	-0.95	-0.19	0.50
_S5	1	145	2	11.16	0.63	-2.05	-0.43	0.42
_S6	1	145	2	16.05	1.02	-2.51	-0.54	0.79
_S7	1	145	2	16.66	0.90	-3.33	-0.72	0.73
_S8	1	145	2	6.21	0.47	-0.57	-0.11	0.30
_S9	1	145	2	7.03	0.31	-1.67	-0.35	0.22

Design

Status : OK

Soil Foundation Interaction

Allowable stress	:	3E03 [Lb/ft2]
Min. safety factor for sliding	:	1.25
Min. safety factor for overturning	:	1.25
Controlling condition	:	S2 - 1

Condition	qmean	qmax	Δmax	Area in compression		Overturning		FS
Footing	[Lb/ft2]	[Lb/ft2]	[in]	[ft2]	(%)	FSx	FSz	slip
S2 - 1	975	1.24E03	0.0874	44.00	100	117.17	15.58	26.27

Bending

Factor ϕ : 0.90
Min rebar ratio : 0.00180

Development length

Axis	Pos.	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
z	Bot.	28.39	16.56	9.00	9.00
x	Bot.	30.98	16.56	9.75	9.75
x	Top	12.00	16.56	9.75	9.75

Axis	Pos.	Condition Footing	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	D8 - 1	-9.66	-198.85	0.10	2.20	0.047	0.049
zz	Bot.	D2 - 1	4.17	198.85	1.97	2.20	0.897	0.021
Zone 1 xx	Top	D1 - 1	0.00	0.00	0.00	0.00	0.000	0.000
Zone 1 xx	Bot.	D8 - 1	0.61	229.09	2.27	2.64	0.859	0.003
Zone 2 xx	Top	D1 - 1	0.00	0.00	0.00	0.00	0.000	0.000
Zone 2 xx	Bot.	D1 - 1	0.00	114.64	1.17	1.32	0.884	0.000
Zone 3 xx	Top	D1 - 1	0.00	0.00	0.00	0.00	0.000	0.000
Zone 3 xx	Bot.	D2 - 1	0.70	229.09	2.27	2.64	0.859	0.003

Shear

Factor ϕ : 0.75
Shear area (plane zz) : 6.88 [ft2]
Shear area (plane xx) : 18.22 [ft2]

Plane	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	D1 - 1	0.00	156.35	0.000
yz	D2 - 1	3.49	56.61	0.082

Punching shear

Factor ϕ : 0.75
Perimeter of critical section (b... : 14.75 [ft]
Punching shear area : 24.89 [ft2]
Perimeter of critical section (b... : 14.75 [ft]
Punching shear area : 24.89 [ft2]

Column	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
column 1	D2 - 1	5.92	785.27	0.010
column 2	D2 - 1	9.30	785.27	0.016

Pedestal design

Reinforcement:

Pedestal	Asreq [in2]	Asprov [in2]	Asreq/Asprov
1	5.76	7.20	0.80
2	5.76	7.20	0.80

Biaxial bending:

Pedestal	Condition Footing	Muxx [Kip*ft]	Muzz [Kip*ft]	ϕ*Mnxx [Kip*ft]	ϕ*Mnzz [Kip*ft]	Mc/(ϕ*Mn)
1	D8 - 1	2.06	-1.75	256.34	-218.04	0.01
2	D8 - 1	0.99	-5.05	64.94	-329.72	0.02

Axial:

Pedestal	Condition Footing	Pu [Kip]	ϕ*Pn [Kip]	Pu/(ϕ*Pn)
1	D8 - 1	-21.33	-978.87	0.02
2	D2 - 1	-24.58	-978.87	0.03

Shear:

Pedestal 1:

S provided : 14.00 [in]
S required : 14.00 [in]

Condition	Dir.	Vu [Kip]	Vc [Kip]	Vs [Kip]	ϕ*Vn [Kip]	Vu/(ϕ*Vn)
D8	x	2.32	58.41	123.40	136.35	0.02
D8	z	1.34	58.41	123.40	136.35	0.01

Pedestal 2:

S provided : 14.00 [in]
S required : 14.00 [in]

Condition	Dir.	Vu [Kip]	Vc [Kip]	Vs [Kip]	ϕ*Vn [Kip]	Vu/(ϕ*Vn)
D8	x	1.10	58.41	123.40	136.36	0.01
D8	z	0.85	58.41	123.40	136.36	0.01

Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed.
* The required flexural reinforcement considers at least the minimum reinforcement
* The design bending moment is calculated at the critical sections located at the support faces
* Only rectangular footings with uniform sections and rectangular columns are considered.
* The nominal shear strength is calculated in critical sections located at a distance d from the support face
* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces
* Transverse reinforcement is not considered in footings
* Values shown in red are not in compliance with a provision of the code
*qprom = Mean compression pressure on soil.
*qmax = Maximum compression pressure on soil.
*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus).
* Mn = Nominal moment strength
* Mu/(ϕ*Mn) = Strength ratio.
* Vn = Nominal shear or punchure force (for footings Vn=Vc).
* Vu/(ϕ*Vn) = Shear or punching shear strength ratio.

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Bearing Plate at Pedestal	Date:	9/21/2026
Fastening point:			

Specifier's comments:

1 Anchor Design

1.1 Input data



Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Specification text:	HILTI ∅ 1 IN HEX HEAD ASTM F 1554 GR. 36 WITH 10 IN EFFECTIVE EMBEDMENT DEPTH PER TECHNICAL DATA , CAST IN PLACE INSTALLATION PER MPII
Effective embedment depth:	h_{ef} = 10.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	grouted standoff; restraint level (anchor plate): 2.00; e_b = 2.000 in.; t = 1.000 in.
	grout compressive strength > f_c' = 3,000 psi
Anchor plate[CBFEM]:	l_x x l_y x t = 16.000 in. x 16.000 in. x 1.000 in.;
Profile:	Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

2" STANDOFF FOR ISOLATOR PADS

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

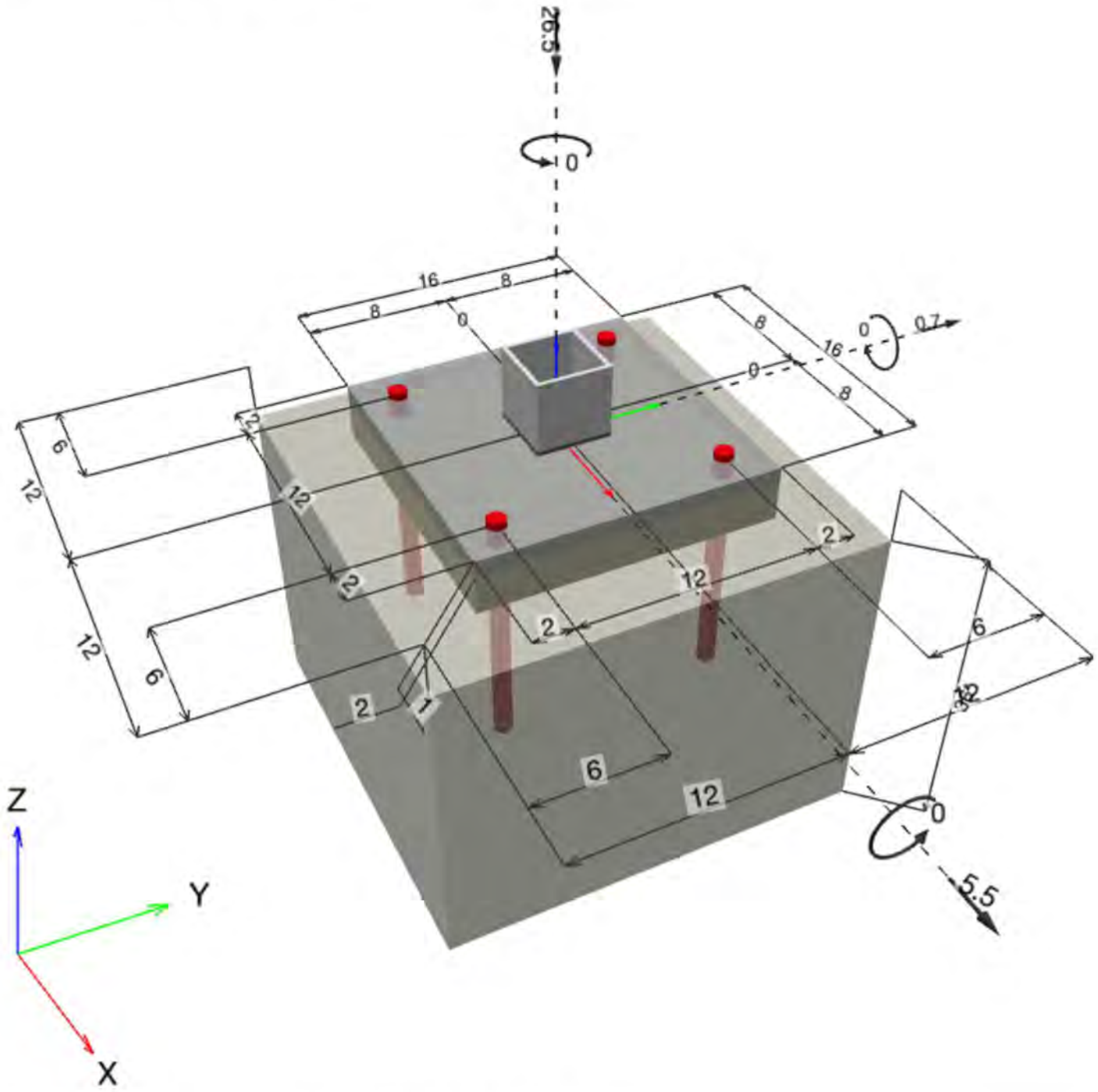
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Bearing Plate at Pedestal	Date:	9/21/2026
Fastening point:			

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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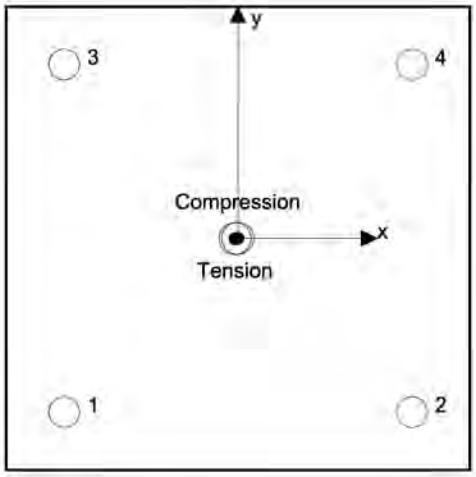
Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Bearing Plate at Pedestal	Date:	9/21/2026
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -26.500; V_x = 5.500; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	92

FORCES FROM RAM ELEMENTS

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.544	1.379	1.367	0.185
2	0.525	1.392	1.384	0.152
3	0.541	1.372	1.362	0.166
4	0.522	1.401	1.387	0.198

Resulting tension force in (x/y)=(-0.108/-0.016): 2.132 [kip]
Resulting compression force in (x/y)=(-0.012/-0.002): 28.774 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	0.544	26.361	3	OK
Pullout Strength*	0.544	19.538	3	OK
Concrete Breakout Failure**	2.132	28.847	8	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Bearing Plate at Pedestal	Date:	9/21/2026
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [kip]
35.148

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
35.148	0.750	26.361	0.544

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.16	1.000	3,000

Calculations

N_p [kip]
27.912

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
27.912	0.700	19.538	0.544

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.000	0.108	0.016	6.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
576.00	144.00	0.982	0.997	1.000	1.000	10.516

Results

N_{cbg} [kip]	$\phi_{concrete}$	ϕN_{cbg} [kip]	N_{ua} [kip]
41.210	0.700	28.847	2.132

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1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.401	10.966	13	OK
Steel failure (with lever arm)*	1.401	1.876	75	OK
Pryout Strength**	5.544	58.891	10	OK
Concrete edge failure in direction y+**	5.544	6.085	92	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [kip]
21.089

Results

V_{sa} [kip]	ϕ_{steel}	ϕ_{eb}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
21.089	0.650	0.800	10.966	1.401

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1.4.2 Steel failure (with lever arm)

V_s^M	$= \frac{\alpha_M \cdot M_s}{L_b}$	bending equation for stand-off
M_s	$= M_s^0 \left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	resultant flexural resistance of anchor
M_s^0	$= (1.2)\ (S)\ (f_{u,min})$	characteristic flexural resistance of anchor
$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$		reduction for tensile force acting simultaneously with a shear force on the anchor
S	$= \frac{\pi (d)^3}{32}$	elastic section modulus of anchor bolt at concrete surface
L_b	$= z + (n)(d_0)$	internal lever arm adjusted for spalling of the surface concrete
ϕV_s^M	$\geq V_{ua}$	ACI 318-19 Table 17.5.2

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [kip]	ϕN_{sa} [kip]	z [in.]	n	d_0 [in.]
2.00	58,000	0.522	26.361	2.500	0.500	1.000

Calculations

M_s^0 [ft.kip]	$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	M_s [ft.kip]	L_b [in.]
0.36815	0.980	0.36086	3.000

Results

V_s^M [kip]	ϕ_{steel}	ϕV_s^M [kip]	V_{ua} [kip]
2.887	0.650	1.876	1.401

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1.4.3 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
576.00	144.00	1.000	1.000	1.000	1.000	10.516

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
84.130	0.700	58.891	5.544

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1.4.4 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	6.000	0.001	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
216.00	162.00	1.000	0.900	1.000	7.245

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
8.693	0.700	6.085	5.544

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.074	0.911	1.000	83	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, ACI318, IS1946, AS 5216:2026, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Engineering calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!

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1.7 Installation data

Profile: Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 1.000 in.

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

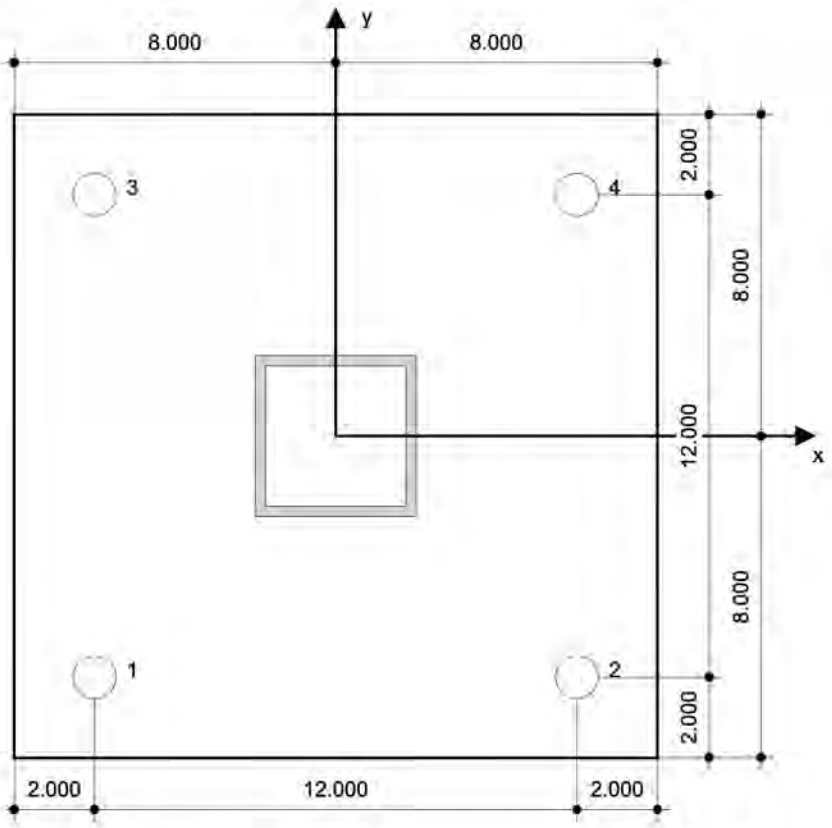
Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Minimum thickness of the base material: 11.172 in.

Hilti ⌀ 1 in Hex Head ASTM F 1554 GR. 36 with 10 in effective embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.000	-6.000	6.000	18.000	6.000	18.000
2	6.000	-6.000	18.000	6.000	6.000	18.000
3	-6.000	6.000	6.000	18.000	18.000	6.000
4	6.000	6.000	18.000	6.000	18.000	6.000

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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 16.000 in x 16.000 in x 1.000 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Hex Head ASTM F 1554 GR. 36 1, h_{ef} = 10.000 in
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 2.000 in (Stand-off with grouting); t = 1.000 in
Profile:	HSS4X4X.25; (L x W x T x FT) = 4.000 in x 4.000 in x 0.250 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 3000; $f_{c,cyl}$ = 3,000 psi; h = 36.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

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2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	16,425	0.00	4,692	0.00	2	7

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.000 kip	0.544 kip
Anchor 2	-0.000 kip	0.525 kip
Anchor 3	-0.000 kip	0.541 kip
Anchor 4	-0.000 kip	0.522 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

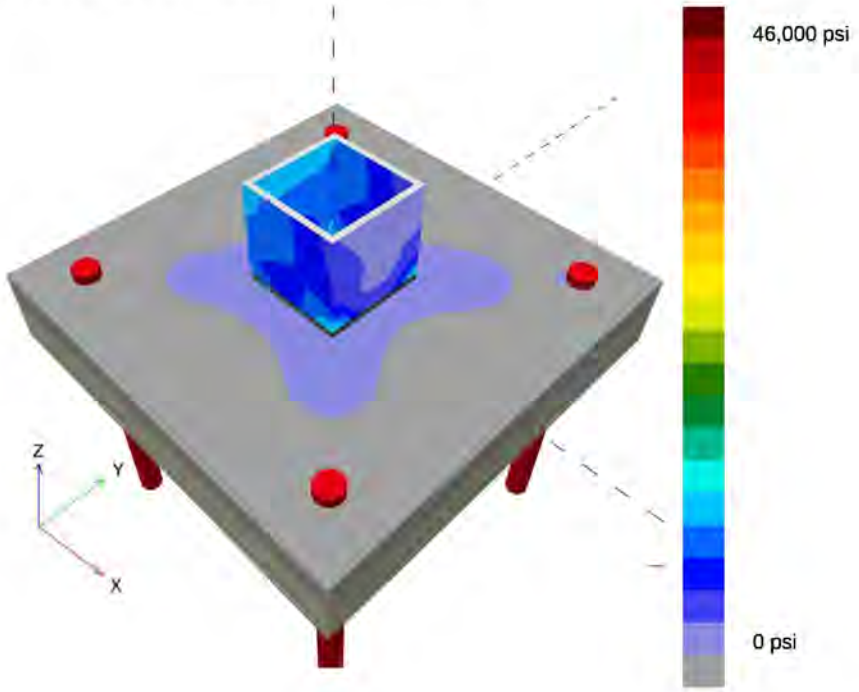
Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	4,692	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	15,626	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	16,425	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	14,963	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	14,790	0.00	OK

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2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.1.2 Plastic strain

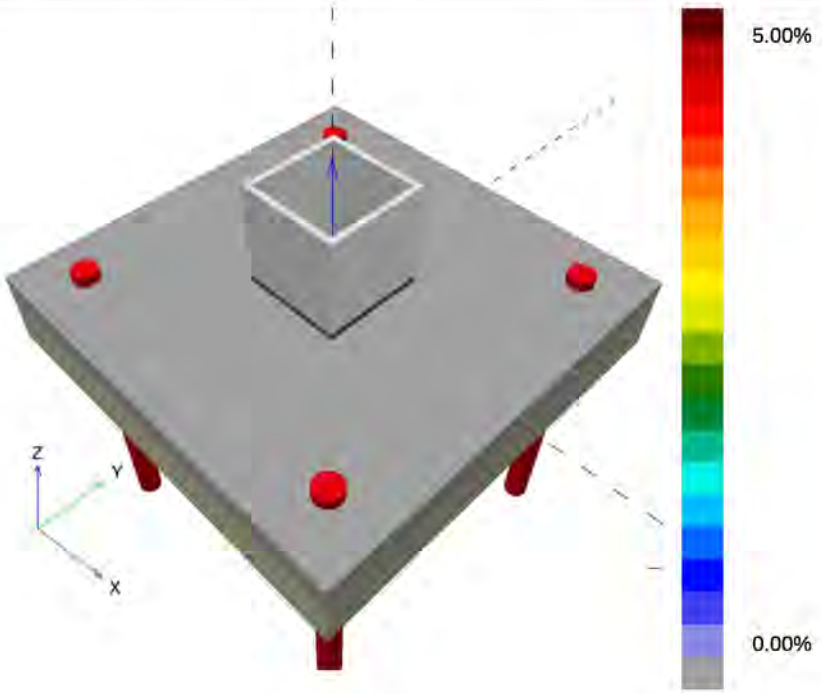
Results below are displayed for the decisive load combination: 1 - Combination 1

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2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u, 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\phi R_n = 0.75\, R_n$

$V \leq \phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	1.487	1.000	58,000	1.000	103.488
Anchor 2	13.553	1.000	58,000	1.000	139.200
Anchor 3	1.484	1.000	58,000	1.000	103.256
Anchor 4	13.610	1.000	58,000	1.000	139.200

Results

	V [kip]	ϕR_n [kip]	Utilization [%]	Status
Anchor 1	1.379	77.616	2	OK
Anchor 2	1.392	104.400	2	OK
Anchor 3	1.372	77.442	2	OK
Anchor 4	1.401	104.400	2	OK

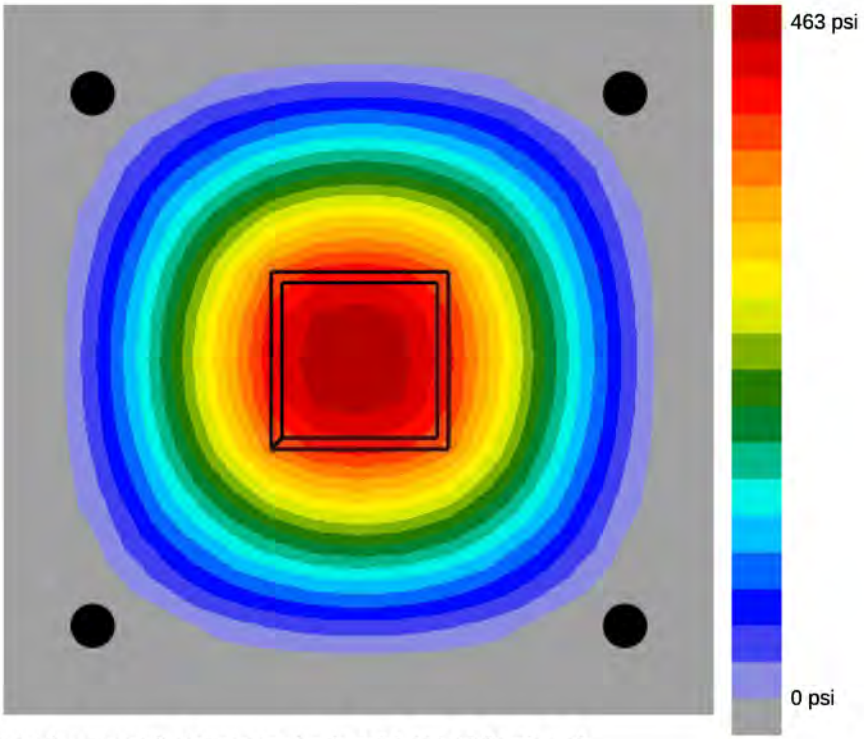
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2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Variables

N [kip]	f_c' [psi]	ϕ	A_1 [in²]	A_2 [in²]
28.774	3,000	0.65	133.06	473.05

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,125	216	7	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
ϕ	Resistance factor
ϕR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	92%	OK
Anchor plate	Combination 1	14%	OK
Concrete	Combination 1	7%	OK
Profile	Combination 1	36%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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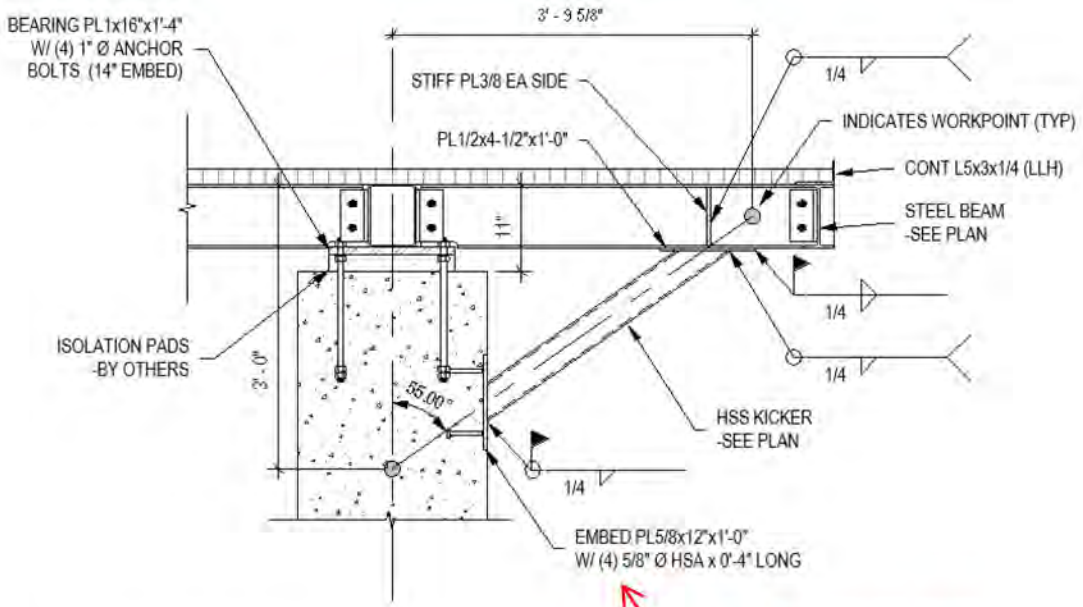
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MECHANICAL EQUIPMENT PLATFORM - EMBED PLATE DESIGN

REACTIONS FROM RAM ELEMENTS MECHANICAL EQUIPMENT PLATFORM



A4 **KICKER DETAIL**

SCALE: 3/4" = 1'-0"

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Specifier's comments:

1 Anchor Design



1.1 Input data

Anchor type and diameter:	**Embed plate D1.1 GR. B 5/8**
Item number:	not available
Specification text:	HILTI ∅ 5/8 IN EMBED PLATE D1.1 GR. B WITH 4.724 IN EFFECTIVE EMBEDMENT DEPTH PER TECHNICAL DATA , CAST IN PLACE INSTALLATION PER MPII
Effective embedment depth:	h_{ef} = 4.724 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Up to first three rows, as applicable (Case 1, 2 from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.625 in.
Anchor plate[CBFEM]:	l_x x l_y x t = 12.000 in. x 12.000 in. x 0.625 in.;
Profile:	Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 24.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

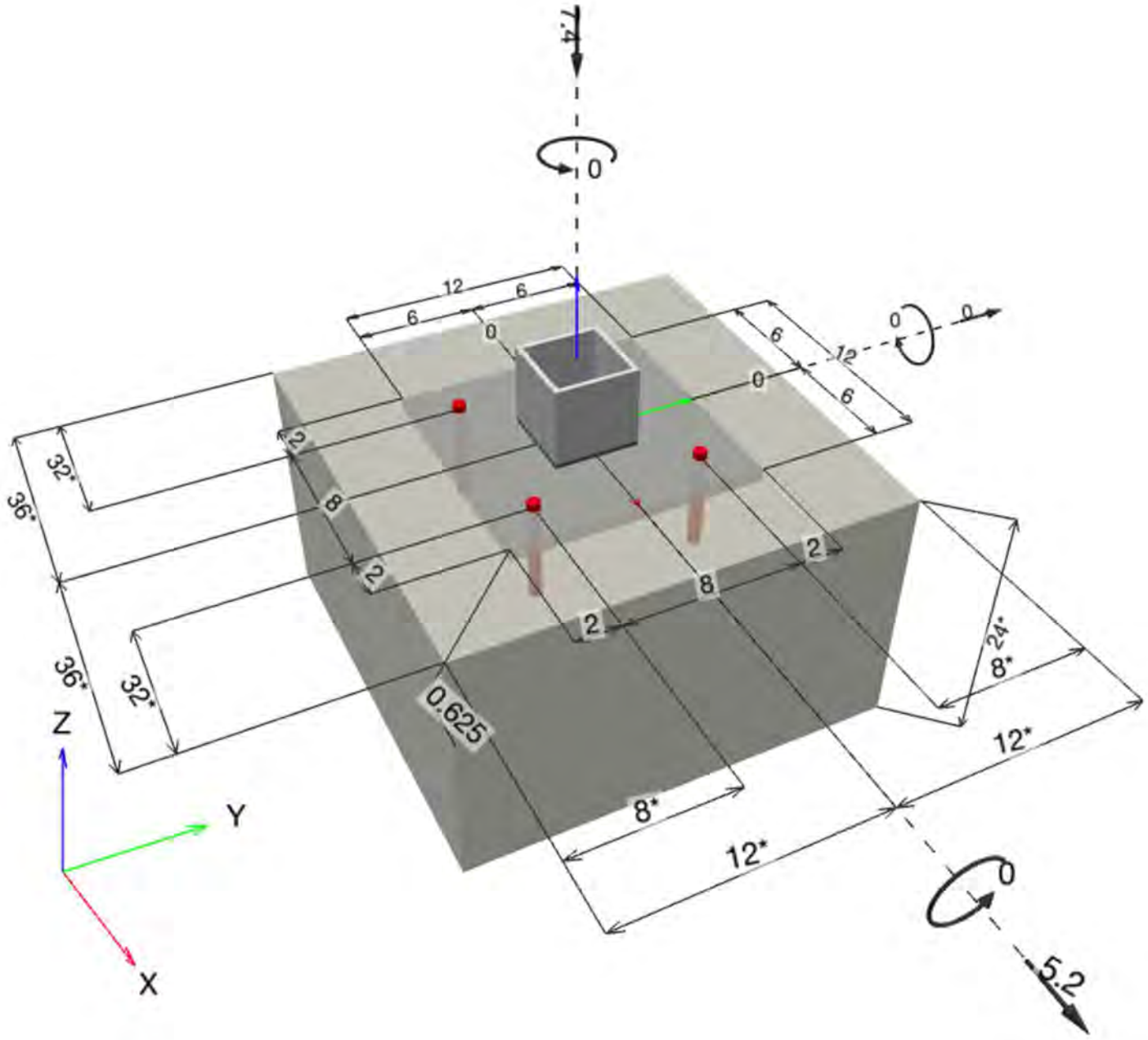
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Geometry [in.] & Loading [kip, ft.kip]



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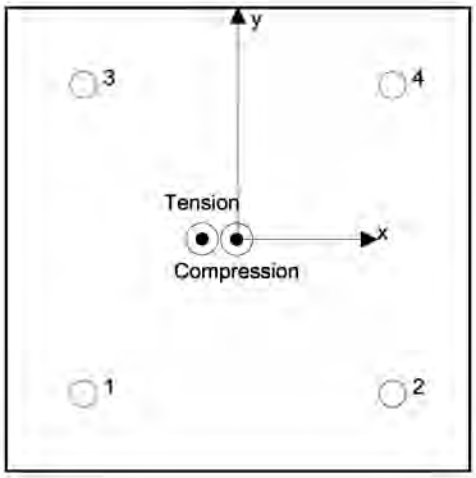
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1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -7.400; V_x = 5.200; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	59

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.085	1.296	1.296	0.004
2	0.052	1.304	1.304	-0.008
3	0.085	1.296	1.296	-0.004
4	0.053	1.304	1.304	0.008

Resulting tension force in (x/y)=(-0.934/0.006): 0.274 [kip]
Resulting compression force in (x/y)=(-0.036/0.001): 7.738 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	0.085	14.966	1	OK
Pullout Strength*	0.085	15.456	1	OK
Concrete Breakout Failure**	0.274	20.414	2	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.31	65,000

Calculations

N_{sa} [kip]
19.955

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.955	0.750	14.966	0.085

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.92	1.000	3,000

Calculations

N_p [kip]
22.080

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
22.080	0.700	15.456	0.085

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1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\Psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\Psi_{c,N}$
4.724	0.934	0.006	8.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\Psi_{ec1,N}$	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	N_b [kip]
491.65	200.88	0.884	0.999	1.000	1.000	13.499

Results

N_{cbg} [kip]	$\phi_{concrete}$	ϕN_{cbg} [kip]	N_{ua} [kip]
29.163	0.700	20.414	0.274

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.304	12.971	11	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	5.200	46.253	12	OK
Concrete edge failure in direction x+**	5.200	8.834	59	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = A_{se,V} f_{uta}$ ACI 318-19 Eq. (17.7.1.2a)
$\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.31	65,000

Calculations

V_{sa} [kip]
19.955

Results

V_{sa} [kip]	ϕ_{steel}	ϕV_{sa} [kip]	V_{ua} [kip]
19.955	0.650	12.971	1.304

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1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.724	0.000	0.000	8.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
491.65	200.88	1.000	1.000	1.000	1.000	13.499

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
66.076	0.700	46.253	5.200

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1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
16.000	8.000	0.000	1.000	24.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
4.724	1.000	0.625	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
576.00	1,152.00	1.000	0.800	1.000	31.549

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
12.620	0.700	8.834	5.200

*Anchor row defined by: Anchor 1, 3; Case 2 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.013	0.589	5/3	42	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, ACI318, IS1946, AS 5216:2026, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!

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1.7 Installation data

Profile: Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 0.625 in.

Anchor type and diameter: Embed plate D1.1 GR. B 5/8

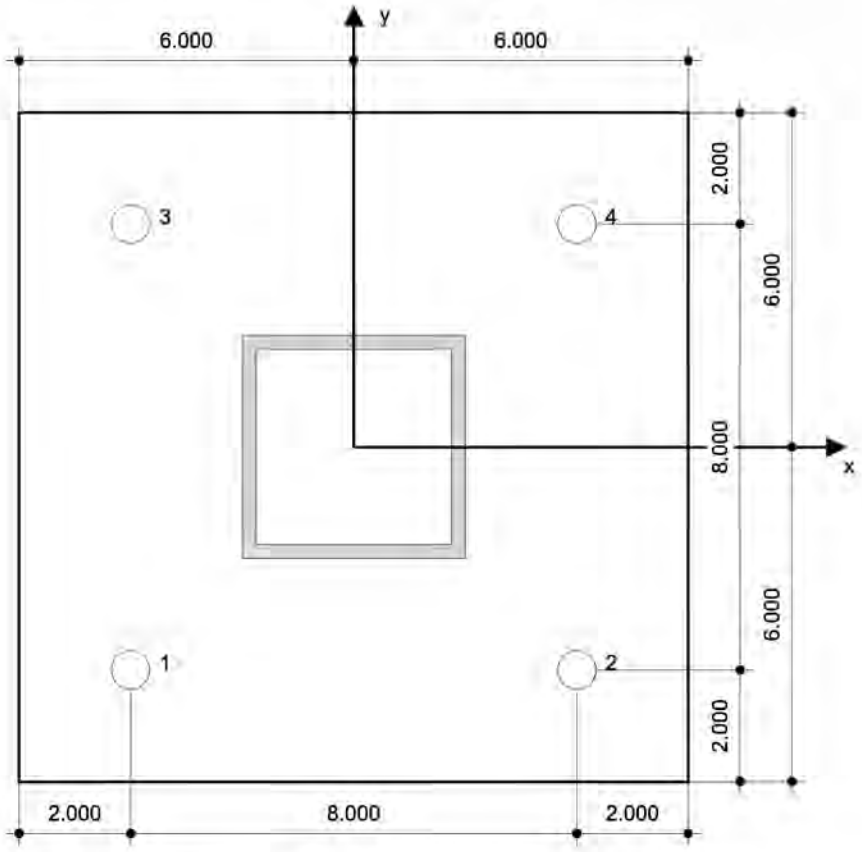
Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Minimum thickness of the base material: 5.537 in.

Hilti ∅ 5/8 in Embed plate D1.1 GR. B with 4.724409 in effective embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.000	-4.000	32.000	40.000	8.000	16.000
2	4.000	-4.000	40.000	32.000	8.000	16.000
3	-4.000	4.000	32.000	40.000	16.000	8.000
4	4.000	4.000	40.000	32.000	16.000	8.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 12.000 in x 12.000 in x 0.625 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Embed plate D1.1 GR. B 5/8, h_{ef} = 4.724 in
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.625 in
Profile:	HSS4X4X.25; (L x W x T x FT) = 4.000 in x 4.000 in x 0.250 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 3000; $f_{c,cyl}$ = 3,000 psi; h = 24.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Kicker Embed	Date:	9/21/2026
Fastening point:			

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	9,270	0.00	3,353	0.00	4	4

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	0.000 kip	0.085 kip
Anchor 2	0.000 kip	0.052 kip
Anchor 3	0.000 kip	0.085 kip
Anchor 4	0.000 kip	0.053 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

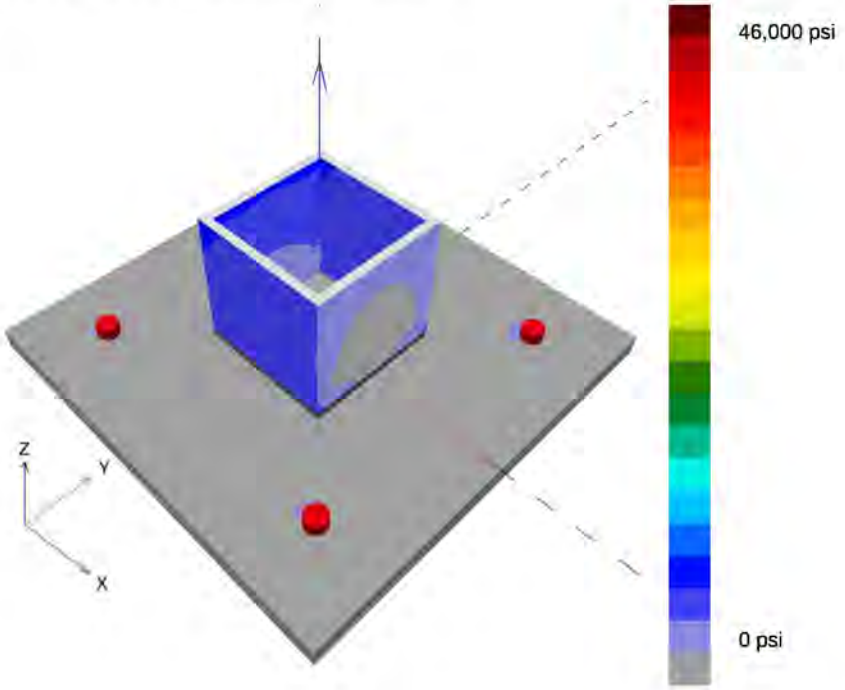
Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	3,353	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	9,270	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	5,519	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	8,667	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	8,667	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Kicker Embed	Date:	9/21/2026
Fastening point:			

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.1.2 Plastic strain

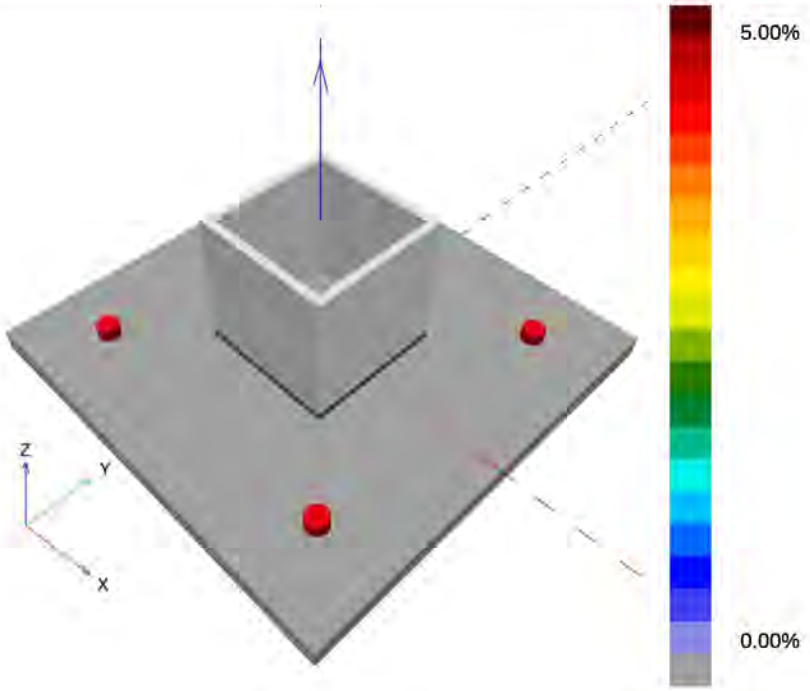
Results below are displayed for the decisive load combination: 1 - Combination 1

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Kicker Embed	Date:	9/21/2026
Fastening point:			

2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u, 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\phi R_n = 0.75\, R_n$

$V \leq \phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	1.656	0.625	58,000	0.625	54.392
Anchor 2	7.313	0.625	58,000	0.625	54.392
Anchor 3	1.656	0.625	58,000	0.625	54.392
Anchor 4	7.313	0.625	58,000	0.625	54.392

Results

	V [kip]	ϕR_n [kip]	Utilization [%]	Status
Anchor 1	1.296	40.794	4	OK
Anchor 2	1.304	40.794	4	OK
Anchor 3	1.296	40.794	4	OK
Anchor 4	1.304	40.794	4	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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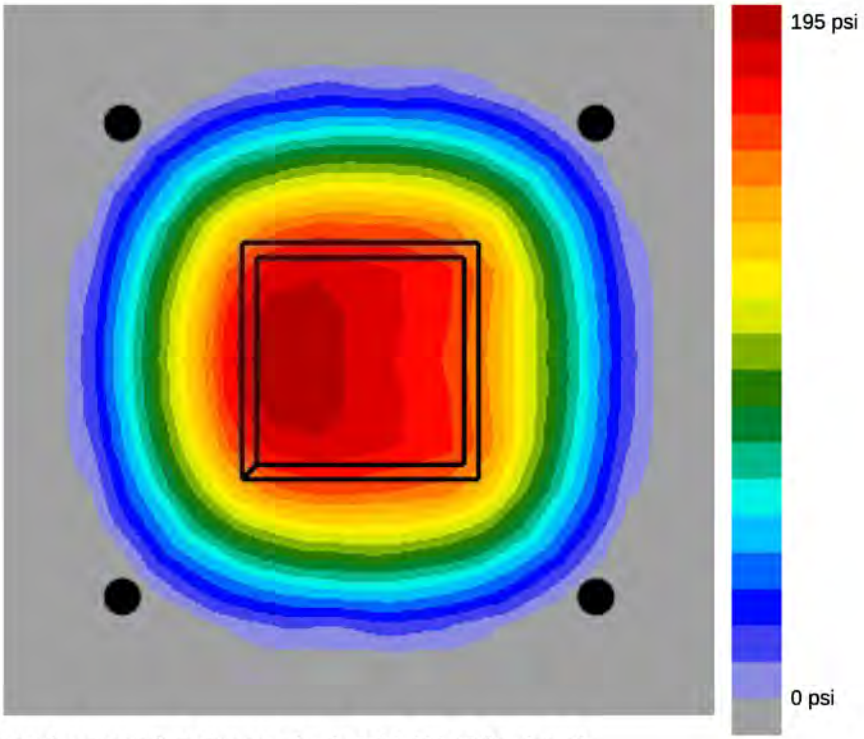
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Design:	Kicker Embed	Date:	9/21/2026
Fastening point:			

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Variables

N [kip]	f_c' [psi]	ϕ	A_1 [in²]	A_2 [in²]
7.738	3,000	0.65	73.54	466.67

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,315	105	4	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
ϕ	Resistance factor
ϕR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	59%	OK
Anchor plate	Combination 1	10%	OK
Concrete	Combination 1	4%	OK
Profile	Combination 1	21%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Kicker Embed	Date:	9/21/2026
Fastening point:			

4 Remarks; Your Cooperation Duties

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Chavez-Grieves
consulting engineers, llc.
A Salas O'Brien Company
4700 Lincoln Road NE, Albuquerque, NM 87109
505.344.4080

SHEET NO.		OF	
PROJECT	PROJECT SUNSHINE		
SUBJECT	SOUND WALL DESIGN		
CLIENT	SALAS O'BRIEN	JOB NO.	2645-01036-02
CALCULATED BY	LAM	DATE	09/21/2026
CHECKED BY	PD	DATE	09/21/2026

Sound Wall Design Criteria

Note: The sound wall is surrounding the mechanical equipment platform but is not attached to it

Dead Load = 5 psf
Wind Load = 45 psf

Height 22'-0"

SonaGuard® (US Patent D865,224)

Perforated Fiberglass Sound-Absorptive Noise Barrier System

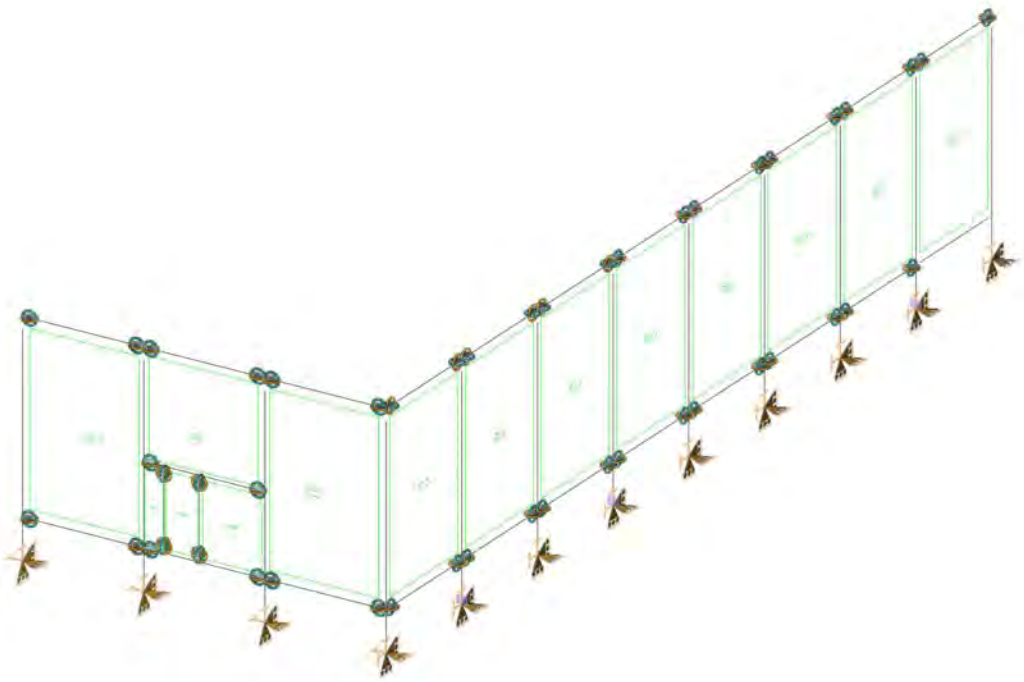
Superior Acoustics — Wide Spans — High Strength



- **Exceptional Acoustical Performance: NRC 1.05 and STC 35**
- **Extreme Strength = Wider Spans = Fewer Foundations = Lower Turnkey Cost**
- **Lightweight: Less than 5 lbs per square foot**
- **Easy to Install and Easy Field Modification**

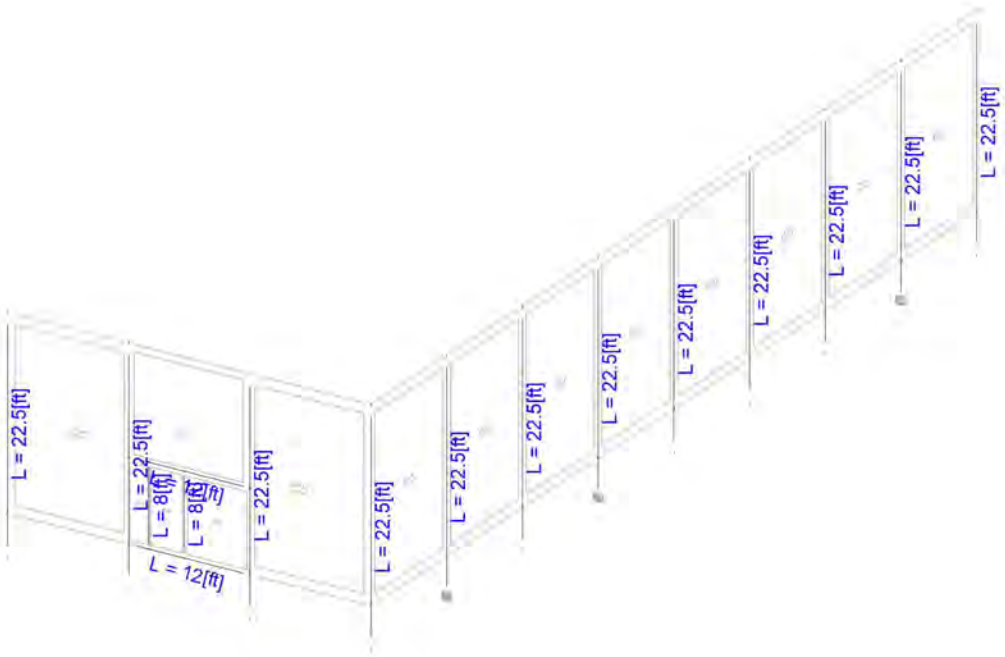
Sound Wall Design

HINGES AND RESTRAINTS



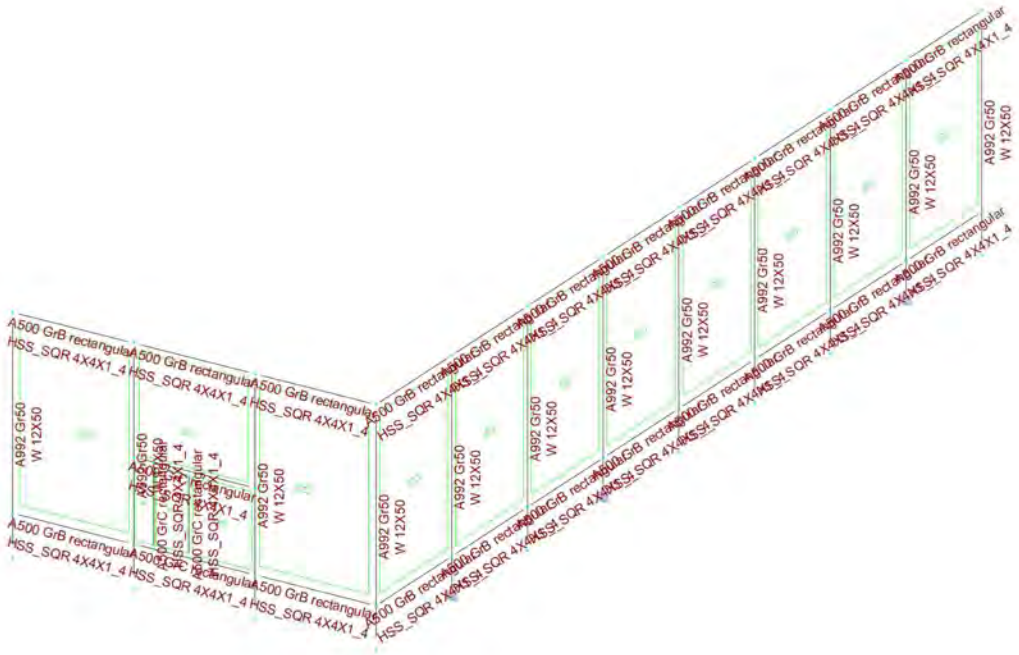
Sound Wall Design

MEMBER LENGTHS



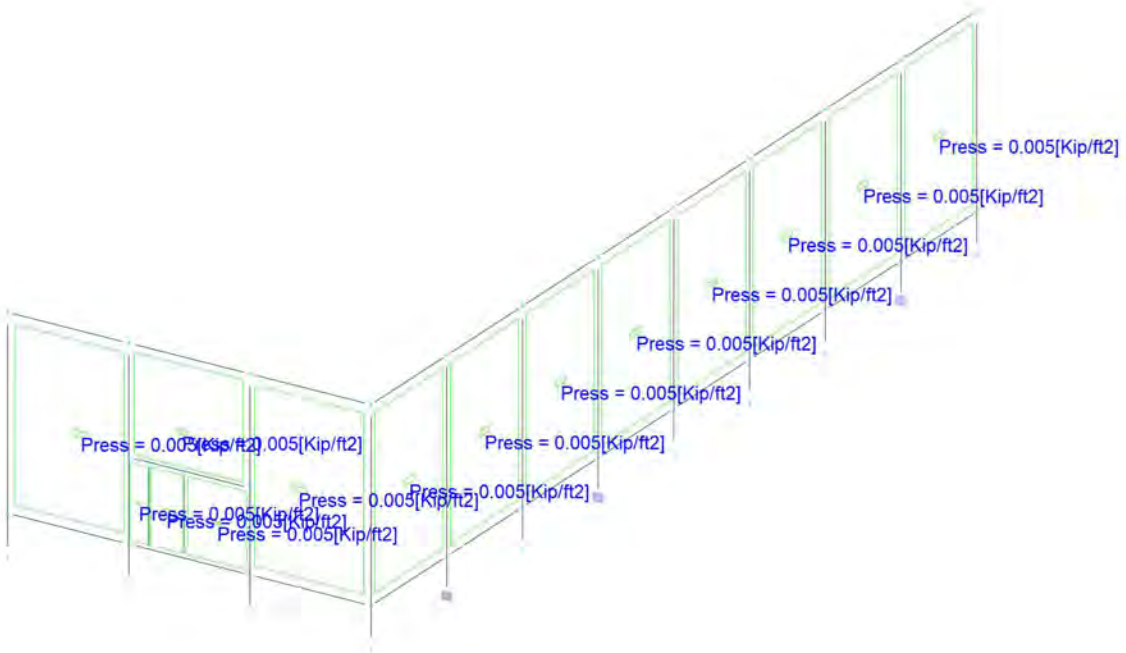
Sound Wall Design

COLUMN SIZE AND MATERIAL



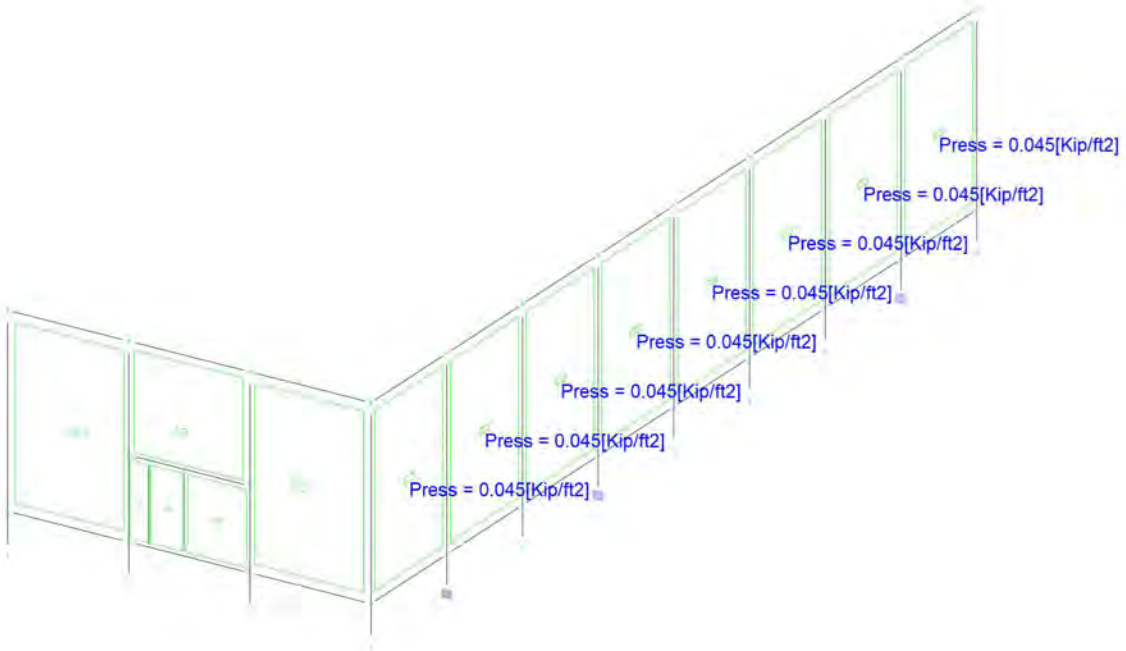
Sound Wall Design

DEAD LOAD = 5 PSF



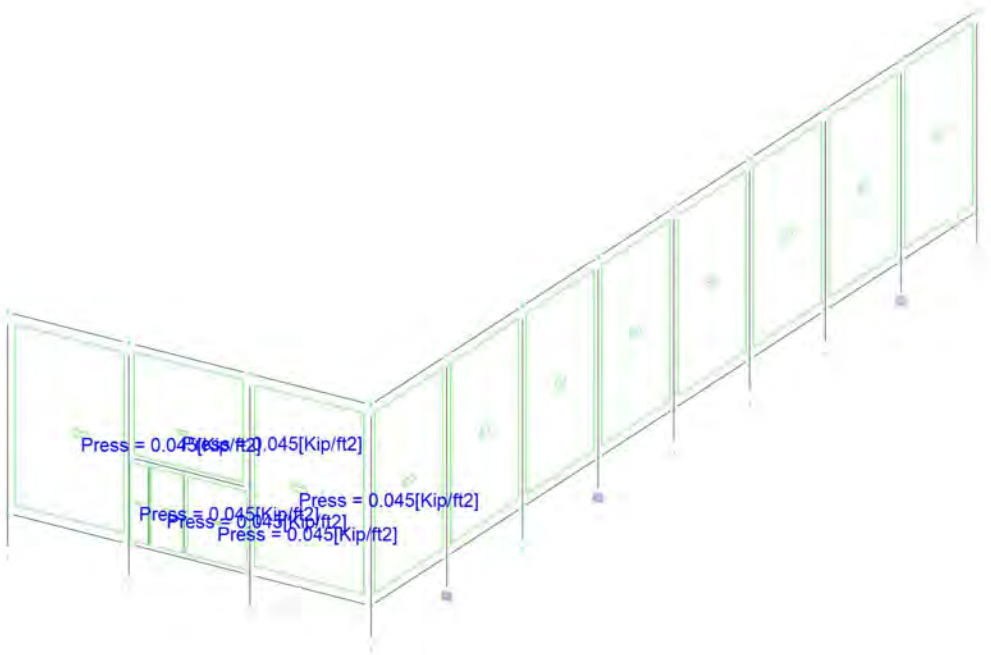
Sound Wall Design

WIND LOAD = 45 PSF



Sound Wall Design

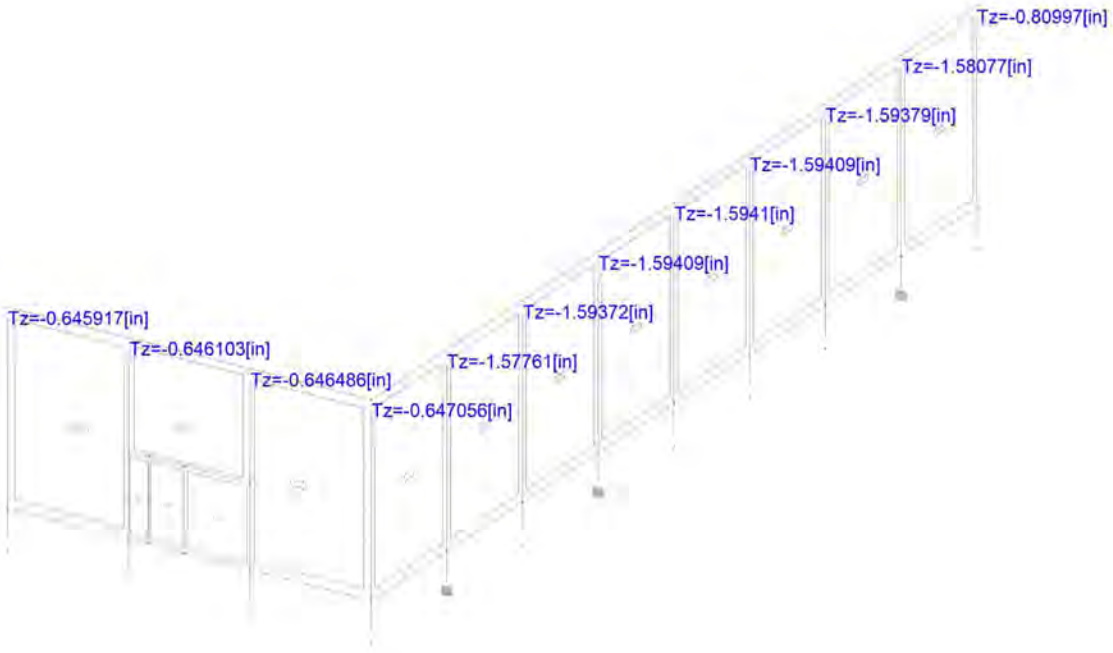
WIND LOAD = 45 PSF



Sound Wall Design

NODE DEFLECTION
DL + 0.6 WL

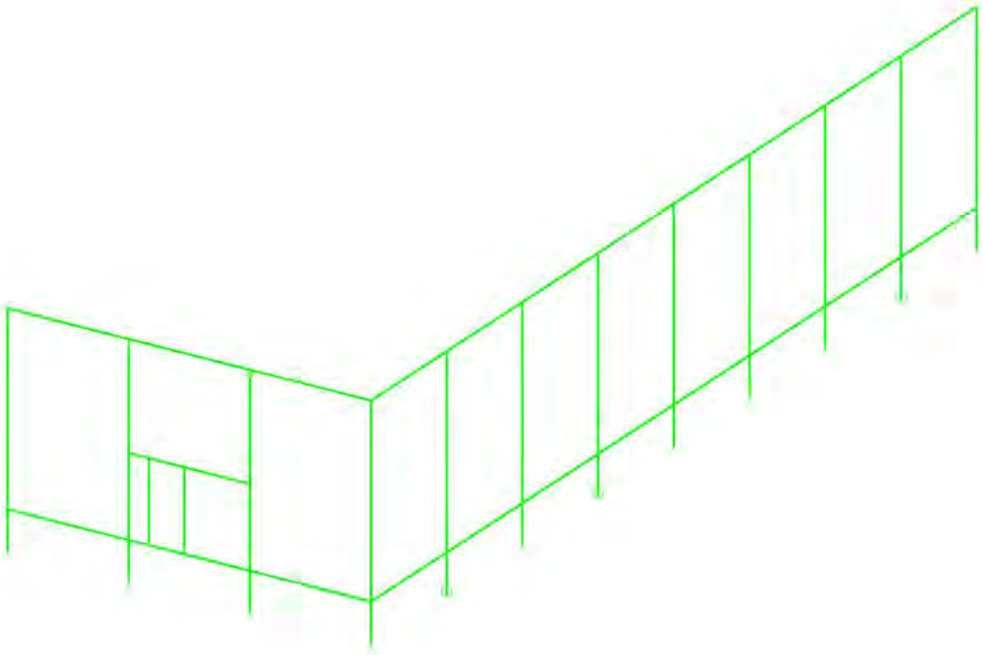
MAX DEFLECTION = L/240 = 2.2 INCHES
IBC CODE 1604.3 - L TO BE TAKEN AS TWICE THE LENGTH



Sound Wall Design

DESIGN CHECK

Design status
- Not designed
- Error on design
- Design O.K.
- With warnings





Sound Wall Design

RAM Elements

Salas O'Brien

Current Date: 9/21/2026 12:03 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\Sound Wall\Auburndale - Sound Wall.retx

Steel Code Check
Comprehensive Report

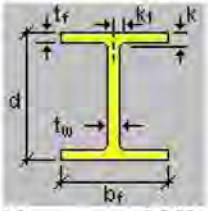
AISC 360-2022 LRFD (Hot-rolled)

Member : 464 (Col - WF - Staggered) - OK

Section information

Section name: W 12X50 **(US)**

Dimensions



bf	=	8.080	[in]	Width
d	=	12.200	[in]	Depth
k	=	1.140	[in]	Distance k
k1	=	0.938	[in]	Distance k1
tf	=	0.640	[in]	Flange thickness
tw	=	0.370	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	14.600	
Moment of Inertia (local axes) (I)	[in4]	391.000	56.300
Moment of Inertia (principal axes) (I')	[in4]	391.000	56.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.175	1.964
Radius of gyration (principal axes) (r')	[in]	5.175	1.964
Saint-Venant torsion constant. (J)	[in4]	1.710	
Section warping constant. (Cw)	[in6]	1880.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	64.200	13.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	64.200	13.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	64.200	13.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	64.200	13.900
Plastic section modulus (local axis) (Z)	[in3]	71.900	21.300
Plastic section modulus (principal axis) (Z')	[in3]	71.900	21.300
Polar radius of gyration. (ro)	[in]	5.535	
Area for shear (Aw)	[in2]	10.340	4.510
Torsional constant. (C)	[in3]	2.511	

Material : A992 Gr50

Sound Wall Design

CANTILEVER COLUMN REPORT

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	22.50

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
22.50	22.50

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
22.50	22.50	22.50	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	657.00 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	657.00	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	730.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	538.43 [Kip]	Reference	:	Cl.E3
Demand	:	3.49 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.31	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	26.81	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	538.43	Cl.E3
Effective length (KL_{33})	[ft]	22.50	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	52.17	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	105.15	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	14.60	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	40.98	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	598.25	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.02			
Capacity	:	174.47 [Kip]	Reference	:	Cl.E3
Demand	:	3.49 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.31	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	26.81	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	174.47	Cl.E3
Effective length (KL_{22})	[ft]	22.50	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	137.49	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	15.14	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	14.60	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	13.28	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	193.86	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	461.20	Cl.E4
Effective length (KL_{11})	[ft]	22.50	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	59.14	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	14.60	
Nominal stress for torsional or flexural-torsional buckling (F_{n11})	[Kip/in2]	35.10	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	512.45	Eq.E4-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.49			
Capacity	:	269.63 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	-132.69 [Kip*ft]	Ctrl Eq.	:	D4 at 0.00%

Sound Wall Design

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.31	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	26.81	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	269.63	Cl.F2.1
Yielding (M_n)	[Kip*ft]	299.58	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	269.63	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	6.94	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	2.25	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	23.86	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	2.28	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	299.58	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.06			
Capacity	:	79.88 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	4.84 [Kip*ft]	Ctrl Eq.	:	D5 at 17.78%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.31	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	26.81	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	79.88	Cl.F6.1
Yielding (M_n)	[Kip*ft]	88.75	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	279.24 [Kip]	Reference	:	Cl.G1
Demand	:	1.23 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	279.24	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	310.27	Eq.G6-1

Shear in minor axis 22

Sound Wall Design

Ratio	:	0.07			
Capacity	:	135.30 [Kip]	Reference	:	Cl.G1
Demand	:	9.99 [Kip]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	135.30	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	135.30	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.49			
Ctrl Eq.	:	D4 at 0.00%	Reference	:	Eq.H1-3a(H1-1b)

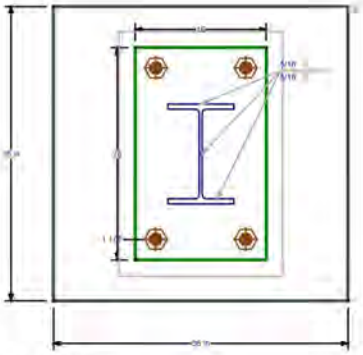
Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending:	--	0.49	Eq.H1-3a(H1-1b)
In-plane available flexural strength (M_{c33})	[Kip*ft]	269.63	Cl.H1.3
In-plane available axial compressive strength (P_c)	[Kip]	538.43	Cl.H1.3
Interaction for doubly symmetric members for out-of-plane bending:	--	0.13	Eq.H1-3b
Out-of-plane available flexural-torsional strength (M_{c33})	[Kip*ft]	176.65	Cl.H1.3
Out-of-plane available axial compressive strength (P_{co})	[Kip]	174.47	Cl.H1.3

Sound Wall Design

COLUMN BASE PLATE REPORT

RAM Elements
Units system: s
Current Date: s

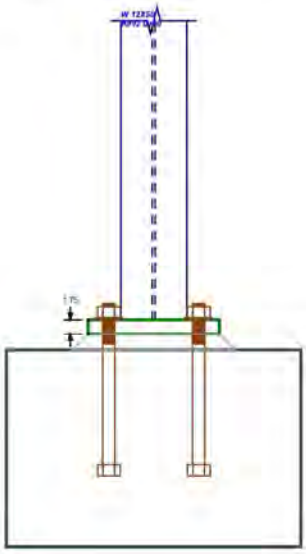
Top view



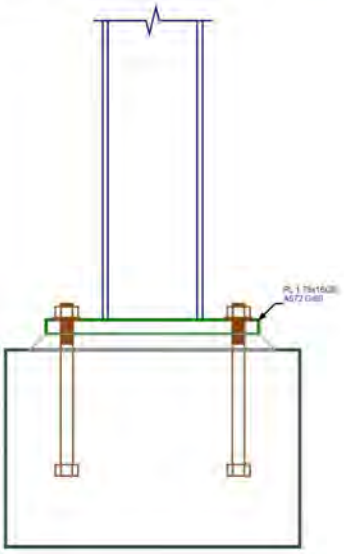
Anchor detailing, Scl.: 1:1



Lateral view



Front view



RAM Elements

Salas O'Brien

Current Date: 9/21/2026 11:50 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\Sound Wall\Auburndale - Sound Wall.retx

Steel Connections Results

Connection: 7 - Fixed biaxial BP

Family: Column - Base (CB)
Type: Base plate
Description: Smart Fixed Biaxial Base Plate 6

Design code: AISC 360-16 LRFD, ACI 318-19

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
D1	-3.49	0.00	0.00	0.00	0.00	Design
D2	-2.99	0.00	66.35	-5.00	0.00	Design
D3	-2.99	0.00	0.00	0.00	0.39	Design
D4	-2.99	0.00	132.69	-9.99	0.00	Design
D5	-2.99	0.00	0.00	0.00	0.79	Design
D6	-2.24	0.00	132.61	-9.99	0.00	Design
D7	-2.24	0.00	0.00	0.00	0.77	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]	1.75	0.25	--	✔	
Weld size	[1/16in]	5	3	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	2.30	1.20	D4	0.52	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	34.45	22.16	D6	0.64	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	34.45	24.61	D6	0.71	DG1 Eq. 3.3.13

Sound Wall Design

Column						
Weld capacity	[Kip/ft]	125.29	48.77	D6	0.39	DG1 p. 35
Elastic method weld shear capacity	[Kip/ft]	83.53	6.04	D4	0.07	Sec. J2.4
Elastic method weld axial capacity	[Kip/ft]	125.29	101.78	D6	0.81	Sec. J2.4
Ratio	**0.81**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	11.00	6.00	--	✓	Sec. 17.9.2
Concrete cover	[in]	6.75	3.00	--	✓	Sec. 20.5.1.3.1
Effective length	[in]	14.97	--	23.03	✓	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	61.13	37.68	D6	0.62	Eq. 17.6.1.2
Pullout of anchor in tension	[Kip]	43.97	37.68	D6	0.86	Sec. 17.5.2
Anchors reinforcement in tension	[Kip]	90.00	75.36	D6	0.84	Sec. 17.5.3
Anchor shear	[Kip]	25.43	2.50	D4	0.10	Eq. 17.7.1.2b, Sec. 17.7.1.2.1
Pryout of anchor in shear	[Kip]	43.59	2.50	D4	0.06	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	80.79	9.99	D4	0.12	Sec. 17.5.2
Anchors reinforcement in shear	[Kip]	39.60	9.99	D4	0.25	Sec. 17.5.3
Interaction of tensile and shear forces		1.20	1.11	D6	0.92	Eq. 17.8.3
Ratio	**0.92**					

Design for minor axis

Base plate (AISC 360-16 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.75	0.25	--	✓	
Weld size	[1/16in]	5	3	--	✓	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	2.30	1.20	D4	0.52	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	34.45	0.22	D1	0.01	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	34.45	24.61	D6	0.71	DG1 Eq. 3.3.13

Column						
Weld capacity	[Kip/ft]	125.29	48.77	D6	0.39	DG1 p. 35
Elastic method weld shear capacity	[Kip/ft]	83.53	0.33	D5	0.00	Sec. J2.4
Elastic method weld axial capacity	[Kip/ft]	125.29	0.00	D1	0.00	Sec. J2.4
Ratio	**0.71**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	11.00	6.00	--	✓	Sec. 17.9.2
Concrete cover	[in]	6.75	3.00	--	✓	Sec. 20.5.1.3.1
Effective length	[in]	14.97	--	23.03	✓	

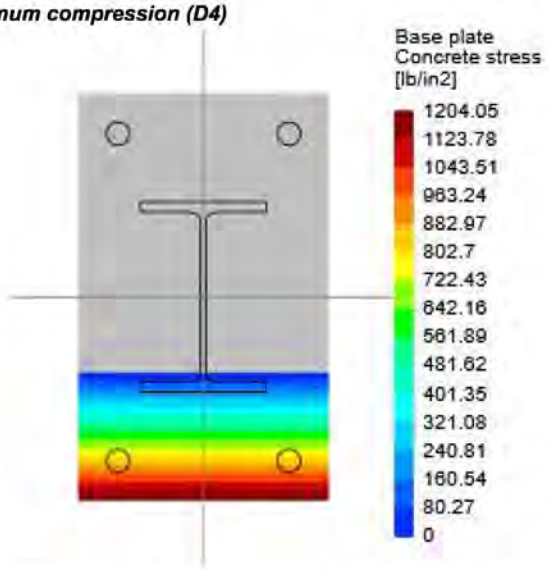
Design Check

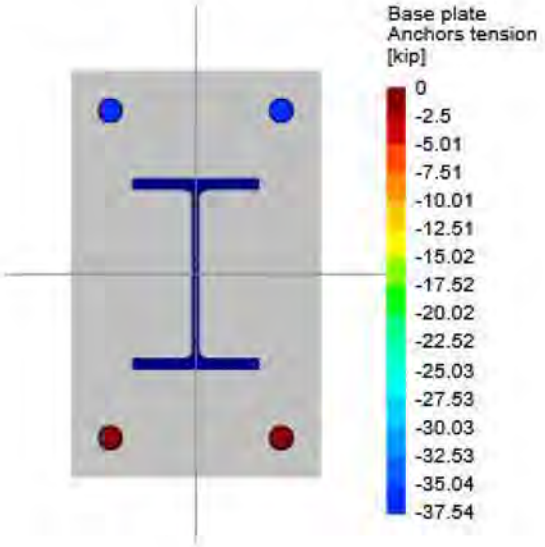
Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	61.13	37.68	D6	0.62	Eq. 17.6.1.2
Pullout of anchor in tension	[Kip]	43.97	37.68	D6	0.86	Sec. 17.5.2
Anchors reinforcement in tension	[Kip]	90.00	75.36	D6	0.84	Sec. 17.5.3
Anchor shear	[Kip]	25.43	2.50	D4	0.10	Eq. 17.7.1.2b, Sec. 17.7.1.2.1
Pryout of anchor in shear	[Kip]	43.59	0.20	D5	0.00	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	80.79	0.79	D5	0.01	Sec. 17.5.2
Anchors reinforcement in shear	[Kip]	39.60	0.79	D5	0.02	Sec. 17.5.3
Interaction of tensile and shear forces		1.20	0.00	D2	0.00	Sec. 17.8.1
Ratio	**0.86**					
Global critical strength ratio	**0.92**					

Biaxial

Sound Wall Design

Maximum compression (D4)





Maximum bearing pressure	1204.05	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	37.54	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	-180.00	[deg]
Neutral axis location	8.10	[in]
Bearing length	8.10	[in]

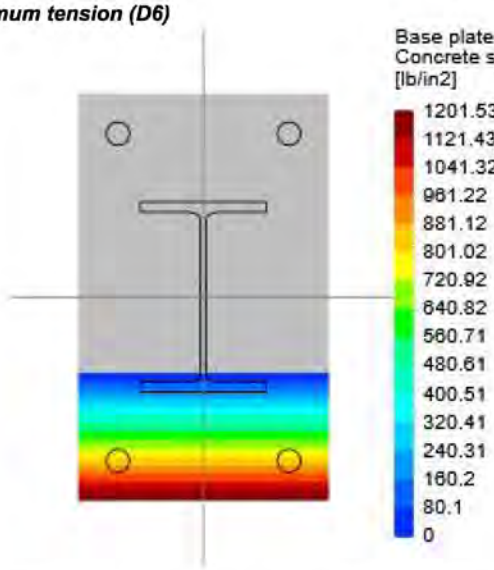
Anchors tensions

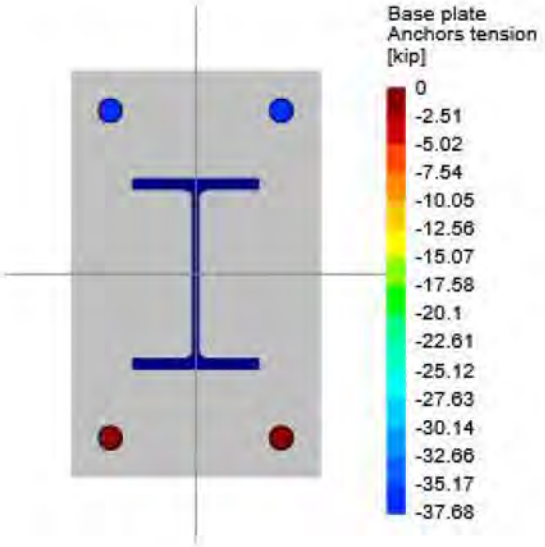
Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-5.50	-10.50	-2.50	0.00
2	-5.50	10.50	-2.50	37.54
3	5.50	10.50	-2.50	37.54
4	5.50	-10.50	-2.50	0.00

Sound Wall Design

COLUMN BASE PLATE REPORT

Maximum tension (D6)





Maximum bearing pressure	1201.53	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	37.68	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	-180.00	[deg]
Neutral axis location	8.07	[in]
Bearing length	8.07	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-5.50	-10.50	-2.50	0.00
2	-5.50	10.50	-2.50	37.68
3	5.50	10.50	-2.50	37.68
4	5.50	-10.50	-2.50	0.00



SHEAR AND TENSION REINFORCEMENT IS REQUIRED. SEE DETAILS

2.95

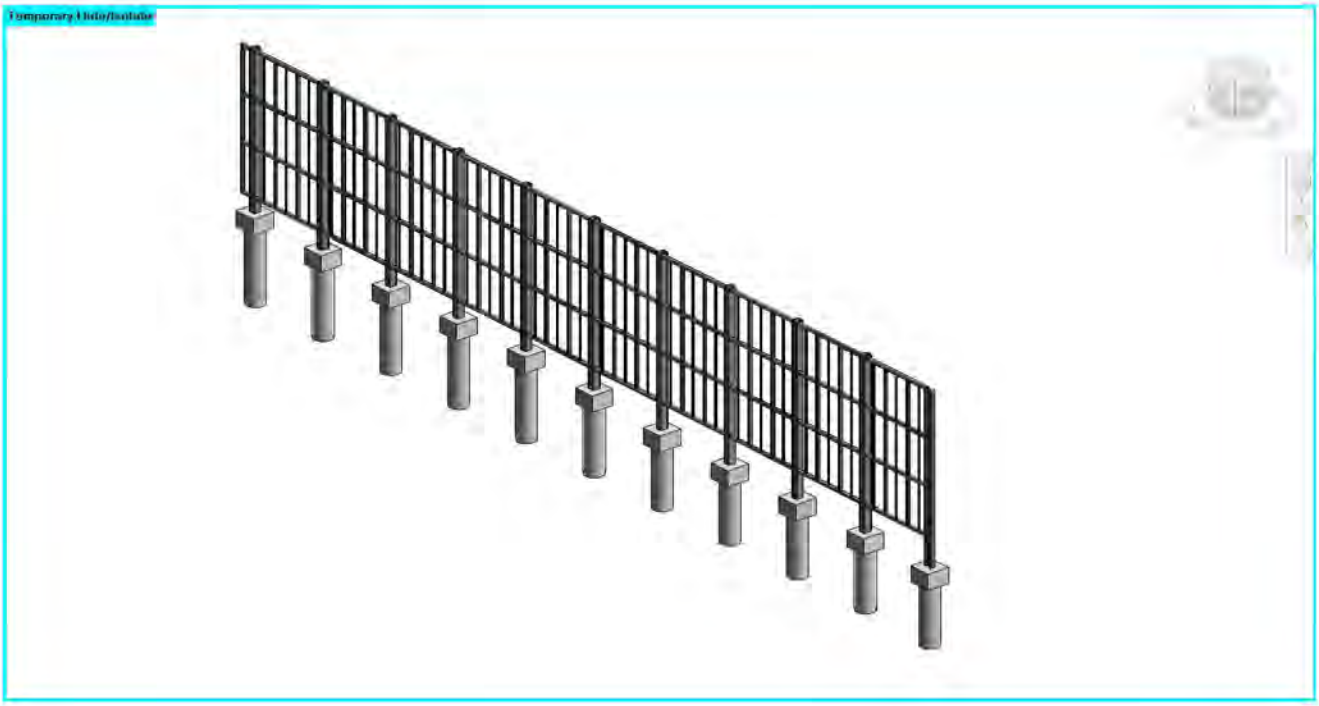
Chavez-Grieves
consulting engineers, llc.
A Salas O'Brien Company
4700 Lincoln Road NE, Albuquerque, NM 87109
505.344.4080

SHEET NO.		OF	
PROJECT	PROJECT SUNSHINE		
SUBJECT	SCREEN WALL DESIGN		
CLIENT	SALAS O'BRIEN	JOB NO.	2645-01036-02
CALCULATED BY	LAM	DATE	09/21/2026
CHECKED BY	PD	DATE	09/21/2026

SCREEN WALL DESIGN

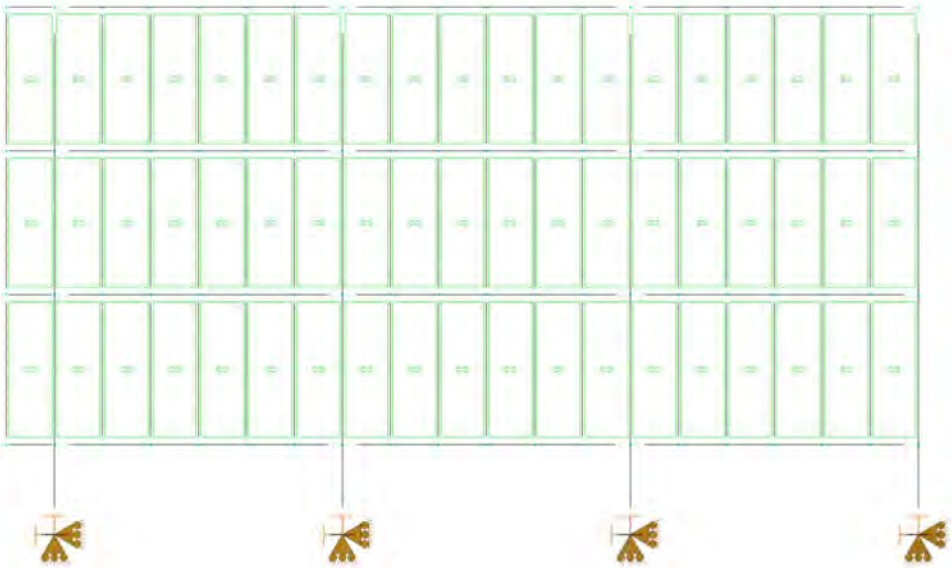
DEAD LOAD = 2 PSF
WIND LOAD = 45 PSF

Note: The screen wall is surrounding the mechanical equipment platform but is not attached to it



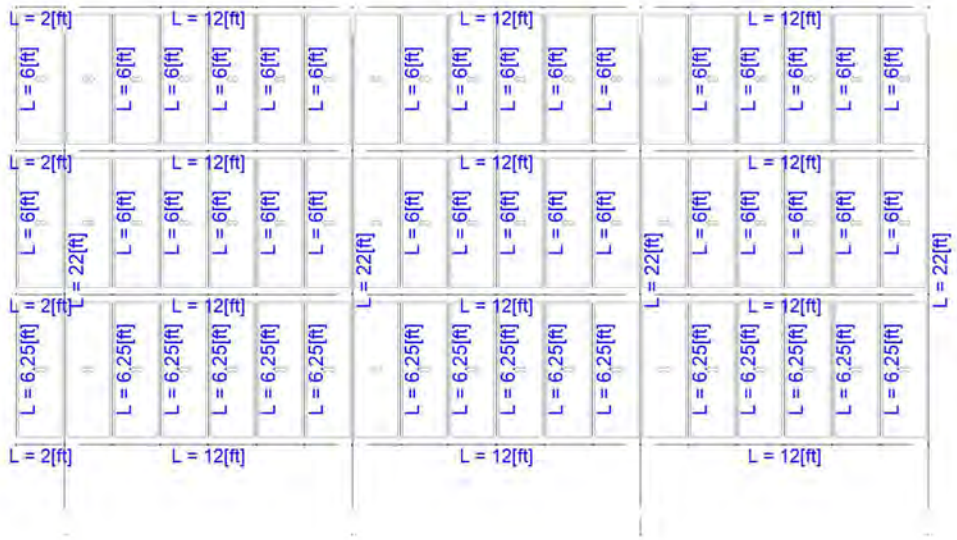
Screen Wall Design

HINGES AND RESTRAINTS



Screen Wall Design

MEMBER LENGTHS



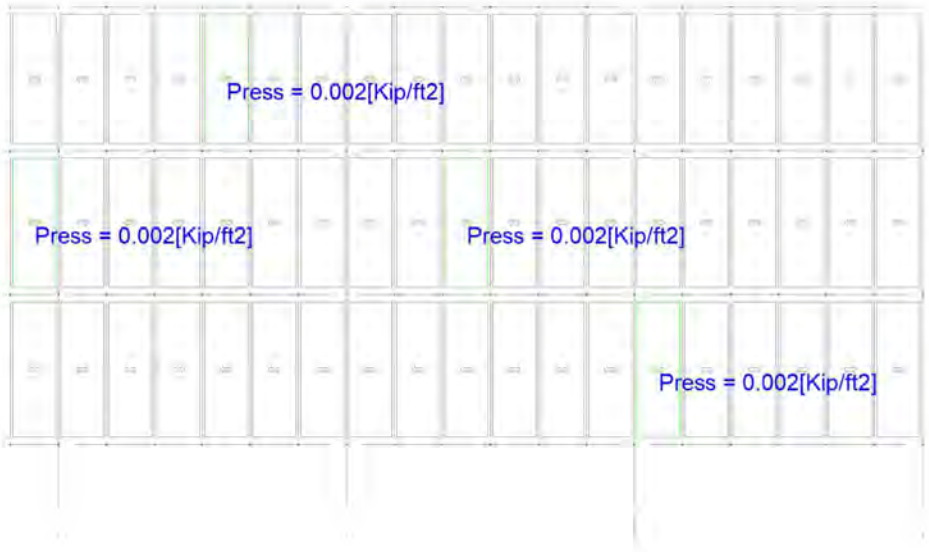
Screen Wall Design

MATERIAL AND SIZES



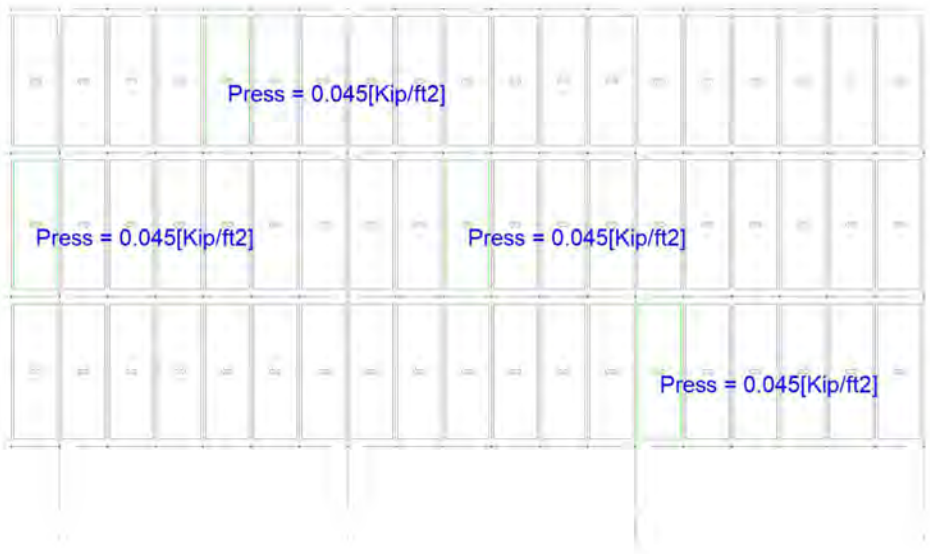
Screen Wall Design

DEAD LOAD = 2 PSF



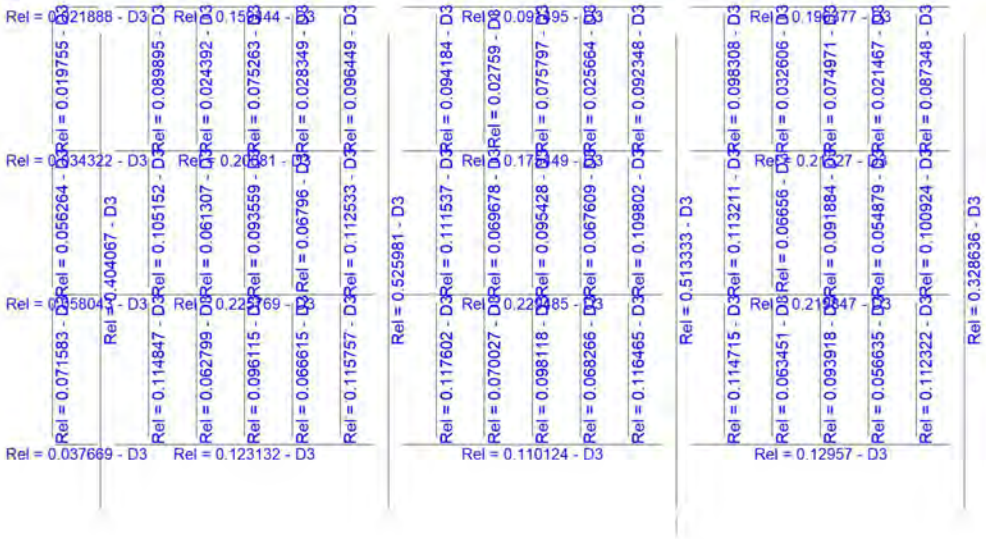
Screen Wall Design

WIND LOAD = 45 PSF



Screen Wall Design

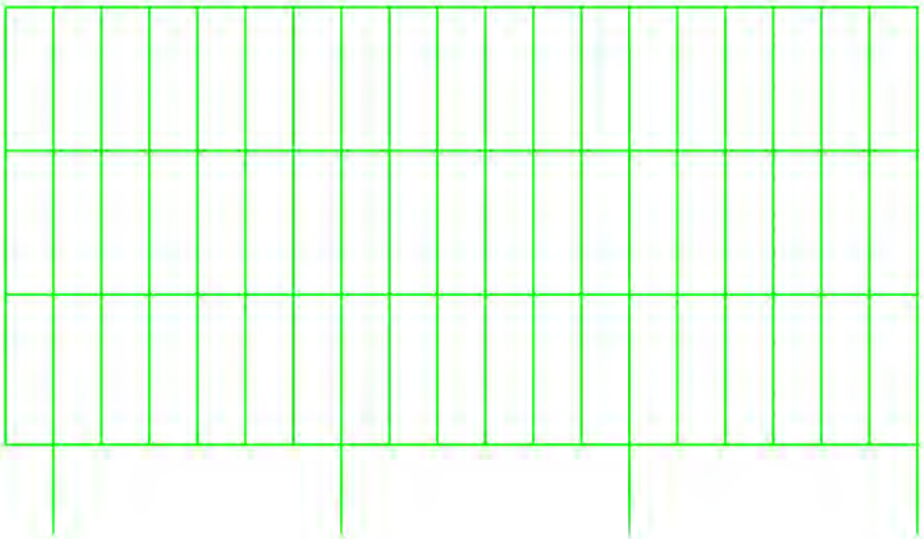
MEMBER DESIGN RATIO



Screen Wall Design

STATUS

Design status
- Not designed
- Error on design
- Design O.K.
- With warnings



Screen Wall Design

RAM Elements

Salas O'Brien

Current Date: 9/21/2026 12:38 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\Screen Wall\Screenwall - Cantilever.retx

Steel Code Check
Comprehensive Report

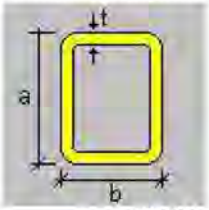
AISC 360-2022 LRFD (Hot-rolled)

Member : 348 (Vert. Member 2) - OK

Section information

Section name: HSS_RECT 12X10X3_8 **(US)**

Dimensions



a	=	12.000	[in]	Height
b	=	10.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	14.600	
Moment of Inertia (local axes) (I)	[in4]	310.000	234.000
Moment of Inertia (principal axes) (I')	[in4]	310.000	234.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.608	4.003
Radius of gyration (principal axes) (r')	[in]	4.608	4.003
Saint-Venant torsion constant. (J)	[in4]	421.000	
Section warping constant. (Cw)	[in6]	35.845	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	51.600	46.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	51.600	46.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	51.600	46.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	51.600	46.900
Plastic section modulus (local axis) (Z)	[in3]	61.100	54.000
Plastic section modulus (principal axis) (Z')	[in3]	61.100	54.000
Polar radius of gyration. (ro)	[in]	6.114	
Area for shear (Aw)	[in2]	6.249	7.645
Torsional constant. (C)	[in3]	78.318	

Material : A500 GrC rectangular

Screen Wall Design

SCREEN WALL
HSS COLUMN REPORT

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	22.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
22.00	22.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
22.00	22.00	22.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	657.00 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	657.00	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	730.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Screen Wall Design

Ratio	:	0.01			
Capacity	:	516.81 [Kip]	Reference	:	Cl.E3
Demand	:	3.87 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	25.65	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.38	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	516.81	Cl.E3
Effective length (KL_{33})	[ft]	22.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	57.29	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	87.20	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	14.60	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	39.33	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	574.24	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.01			
Capacity	:	478.06 [Kip]	Reference	:	Cl.E3
Demand	:	3.87 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	25.65	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.38	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	478.06	Cl.E3
Effective length (KL_{22})	[ft]	22.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	65.94	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	65.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	14.60	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	36.38	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	531.17	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.51			
Capacity	:	229.13 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-116.75 [Kip*ft]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	25.65	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Screen Wall Design

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	31.38	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	229.13	Cl.F7.1
Yielding (M_n)	[Kip*ft]	254.58	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	185.08 [Kip*ft]	Reference	: Cl.F7.2
Demand	:	-0.07 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	25.65	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	31.38	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	202.50	Cl.F7.1
Yielding (M_n)	[Kip*ft]	225.00	Eq.F7-1
Factored web local buckling strength about a geometric axis(ϕM_n):	[Kip*ft]	185.08	Cl.F7.2
Local web buckling (M_n)	[Kip*ft]	205.65	Eq.F7-2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	168.73 [Kip]	Reference	: Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	: D1 at 72.73%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	168.73	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	187.48	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.05		
Capacity	:	206.42 [Kip]	Reference	: Cl.G1
Demand	:	9.49 [Kip]	Ctrl Eq.	: D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	206.42	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	229.36	Eq.G4-1

Screen Wall Design

Torsion Design ✓

Torsion

Ratio	:	0.02			
Capacity	:	176.21 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-3.03 [Kip*ft]	Ctrl Eq.	:	D3 at 17.05%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	176.21	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	195.79	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

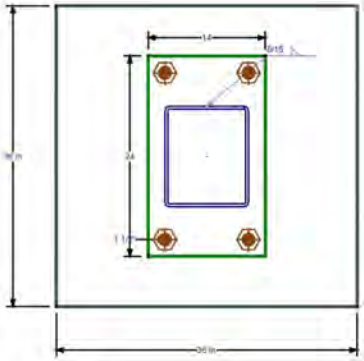
Ratio	:	0.51			
Ctrl Eq.	:	D3 at 0.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.51	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	229.13	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	185.08	Cl.H1.1
Available axial strength (P_c)	[Kip]	478.06	Cl.H1.1

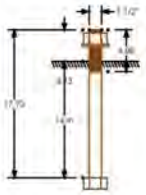
Screen Wall Design

SCREEN WALL BASE PLATE REPORT

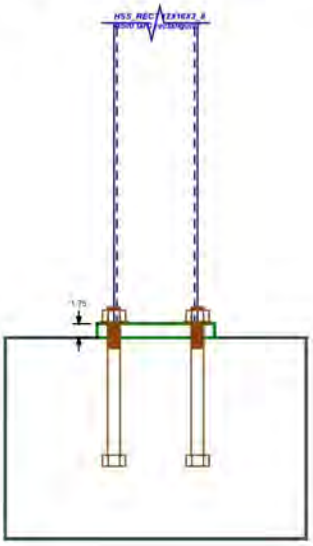
Top view



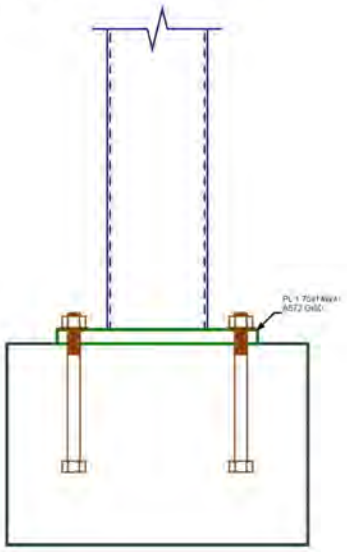
Anchor detailing, Scl.: 1:1



Lateral view



Front view



Screen Wall Design

SCREEN WALL BASE PLATE REPORT

RAM Elements

Salas O'Brien

Current Date: 9/21/2026 12:32 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\Screen Wall\Screenwall - Cantilever.retx

Steel Connections Results

Connection: 1 - Fixed biaxial BP

Family: Column - Base (CB)
Type: Base plate
Description: Smart Fixed Biaxial Base Plate 1

Design code: AISC 360-16 LRFD, ACI 318-19

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
D1	-3.87	0.07	0.00	0.00	0.01	Design
D2	-3.32	0.06	58.38	-4.75	0.01	Design
D3	-3.32	0.06	116.75	-9.49	0.01	Design
D4	-2.49	0.04	116.68	-9.49	0.01	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]	1.25	0.25	--	✓	
Weld size	[1/16in]	8	3	--	✓	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	2.49	1.30	D3	0.52	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	34.45	21.95	D4	0.64	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	34.45	24.40	D4	0.71	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	200.46	85.78	D4	0.43	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	133.64	116.09	D4	0.87	Sec. J2.4

Ratio	0.87

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	10.00	6.00	--	✓	Sec. 17.9.2
Concrete cover	[in]	7.25	3.00	--	✓	Sec. 20.5.1.3.1
Effective length	[in]	14.97	--	23.03	✓	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	61.13	35.74	D4	0.58	Eq. 17.6.1.2
Pullout of anchor in tension	[Kip]	43.97	35.74	D4	0.81	Sec. 17.5.2
Anchors reinforcement in tension	[Kip]	118.80	71.46	D4	0.60	Sec. 17.5.3
Anchor shear	[Kip]	31.79	2.37	D4	0.07	Eq. 17.7.1.2b
Pryout of anchor in shear	[Kip]	45.41	2.37	D4	0.05	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	79.63	9.49	D4	0.12	Sec. 17.5.2
Anchors reinforcement in shear	[Kip]	29.70	9.49	D4	0.32	Sec. 17.5.3
Interaction of tensile and shear forces		1.20	1.13	D4	0.94	Eq. 17.8.3
Ratio	**0.94**					

Design for minor axis

Base plate (AISC 360-16 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.25	0.25	--	✓	
Weld size	[1/16in]	8	3	--	✓	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	2.49	1.30	D3	0.52	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	34.45	19.12	D3	0.55	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	34.45	24.40	D4	0.71	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	200.46	85.78	D4	0.43	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	133.64	0.01	D1	0.00	Sec. J2.4
Ratio	**0.71**					

Screen Wall Design

Anchors

Geometric Considerations

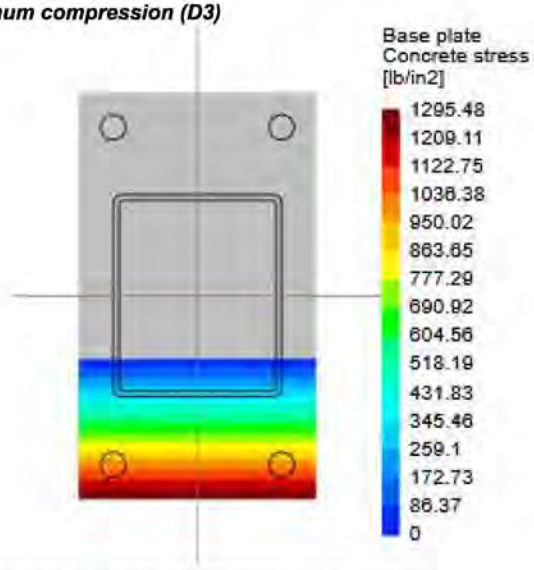
Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	10.00	6.00	--	✔	Sec. 17.9.2
Concrete cover	[in]	7.25	3.00	--	✔	Sec. 20.5.1.3.1
Effective length	[in]	14.97	--	23.03	✔	

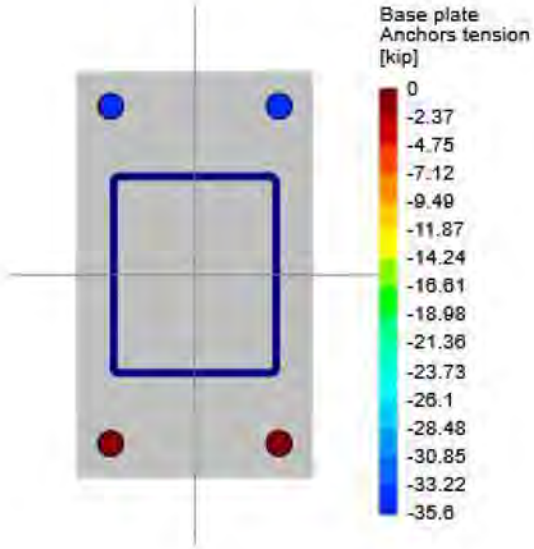
Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	61.13	35.74	D4	0.58	Eq. 17.6.1.2
Pullout of anchor in tension	[Kip]	43.97	35.74	D4	0.81	Sec. 17.5.2
Anchors reinforcement in tension	[Kip]	118.80	71.46	D4	0.60	Sec. 17.5.3
Anchor shear	[Kip]	31.79	2.37	D4	0.07	Eq. 17.7.1.2b
Pryout of anchor in shear	[Kip]	45.41	0.00	D1	0.00	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	79.63	0.01	D1	0.00	Sec. 17.5.2
Anchors reinforcement in shear	[Kip]	29.70	0.01	D1	0.00	Sec. 17.5.3
Interaction of tensile and shear forces		1.20	0.00	D2	0.00	Sec. 17.8.1
Ratio	**0.81**					
Global critical strength ratio	**0.94**					

Biaxial

Maximum compression (D3)





Maximum bearing pressure	1295.48	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	35.60	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	-179.91	[deg]
Neutral axis location	8.23	[in]
Bearing length	8.23	[in]

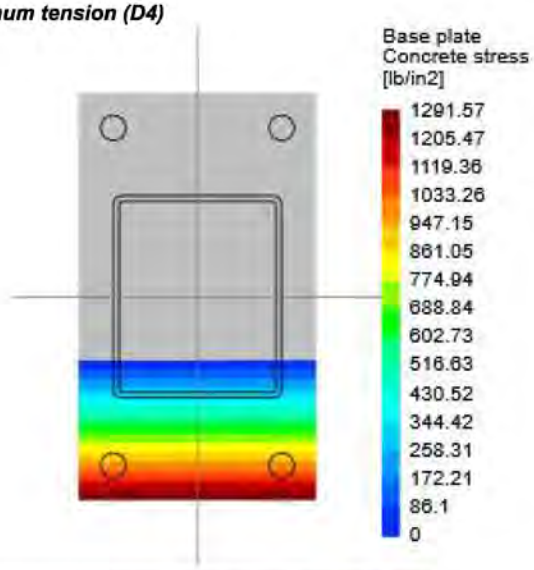
Anchors tensions

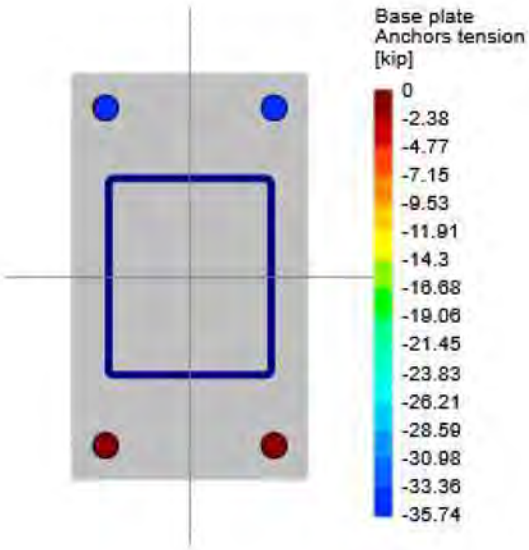
Screen Wall Design

SCREEN WALL BASE PLATE REPORT

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-5.00	-10.00	-2.37	0.00
2	-5.00	10.00	-2.37	35.60
3	5.00	10.00	-2.37	35.55
4	5.00	-10.00	-2.37	0.00

Maximum tension (D4)





Maximum bearing pressure	1291.57	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	35.74	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	-179.93	[deg]
Neutral axis location	8.20	[in]
Bearing length	8.20	[in]

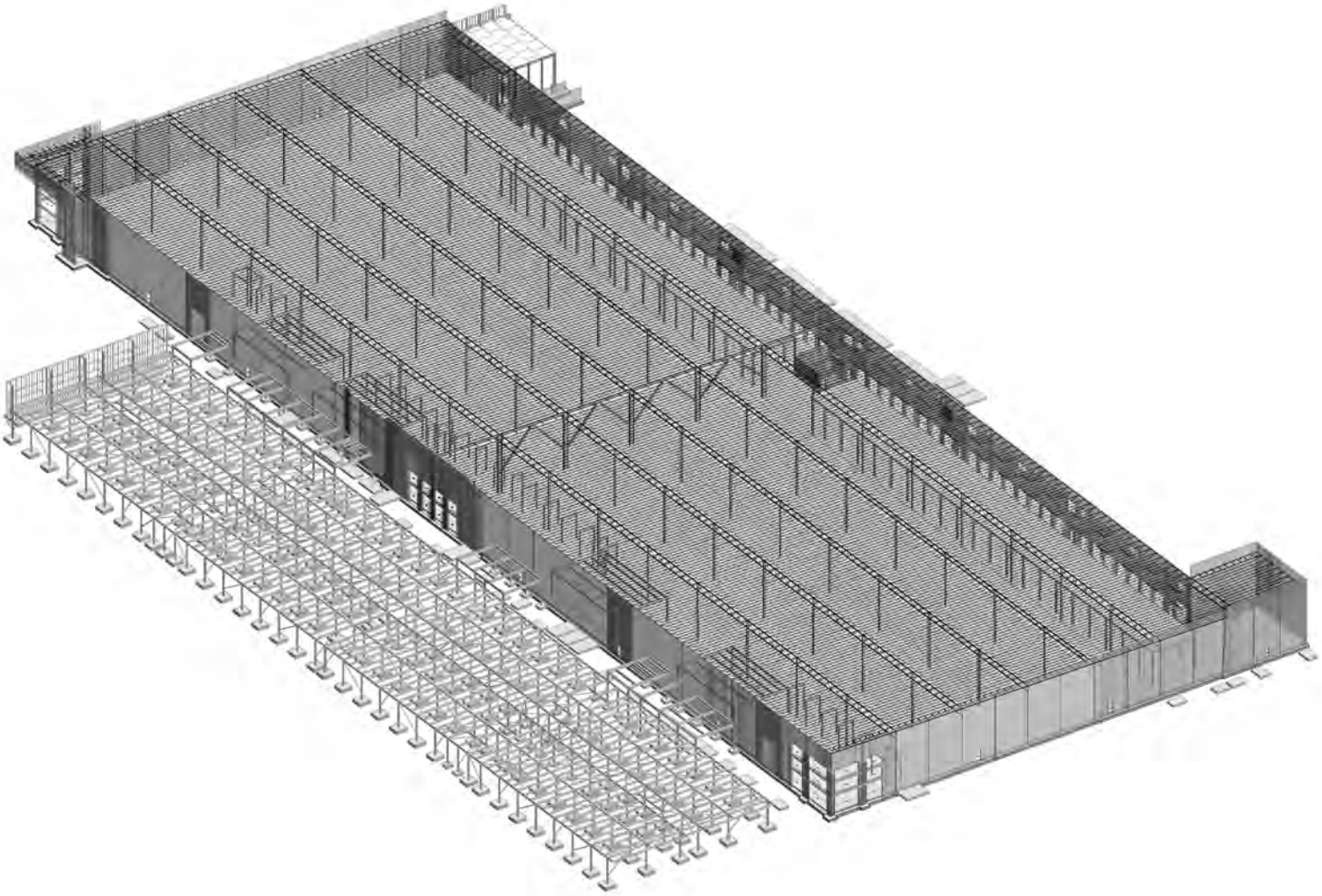
Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-5.00	-10.00	-2.37	0.00
2	-5.00	10.00	-2.37	35.74
3	5.00	10.00	-2.37	35.71
4	5.00	-10.00	-2.37	0.00



ANCHOR REINFORCEMENT IS REQUIRED -SEE DETAIL

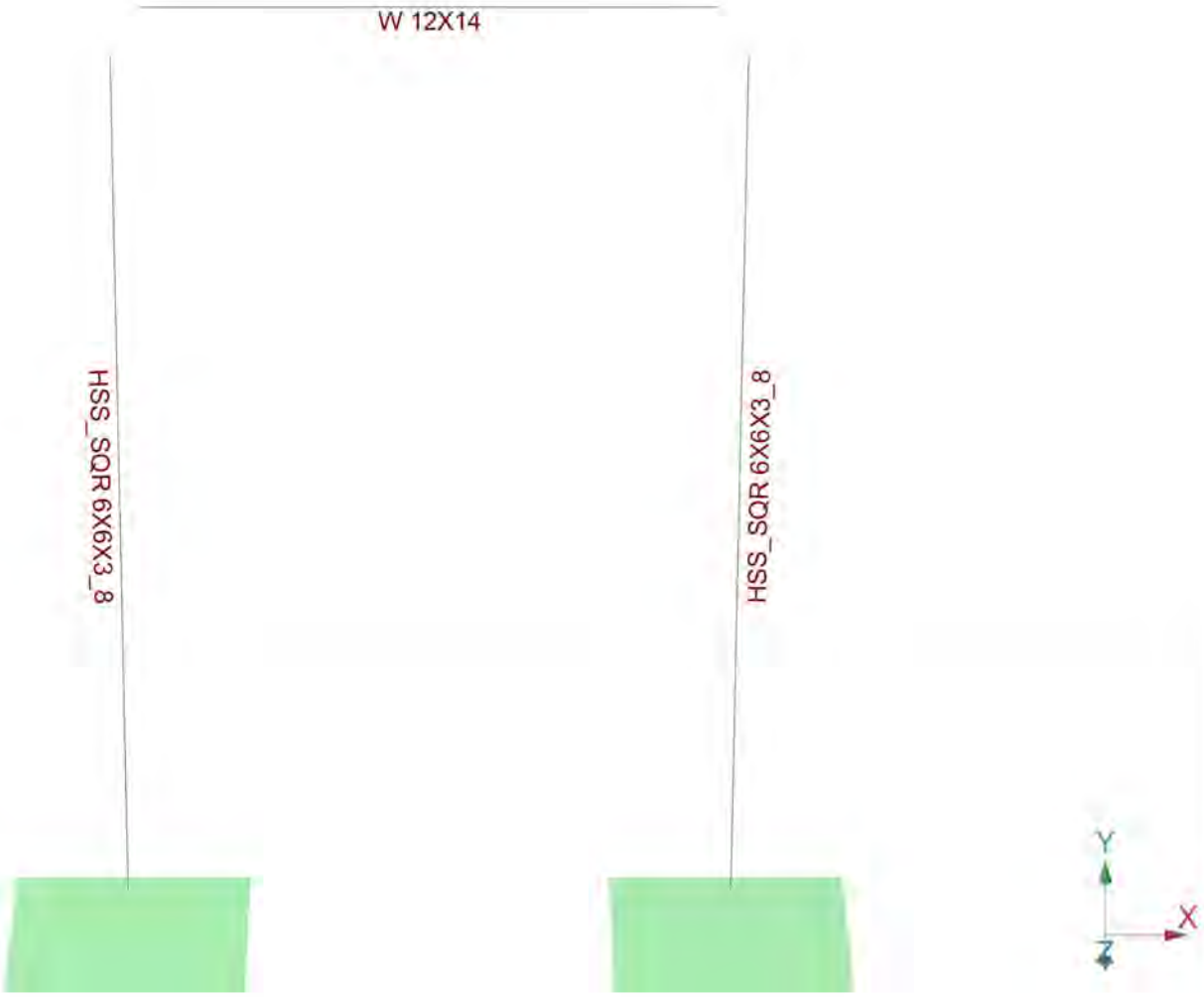
Chapter 3 - Electrical Cable Tray and Mechanical Pipe Support Frames



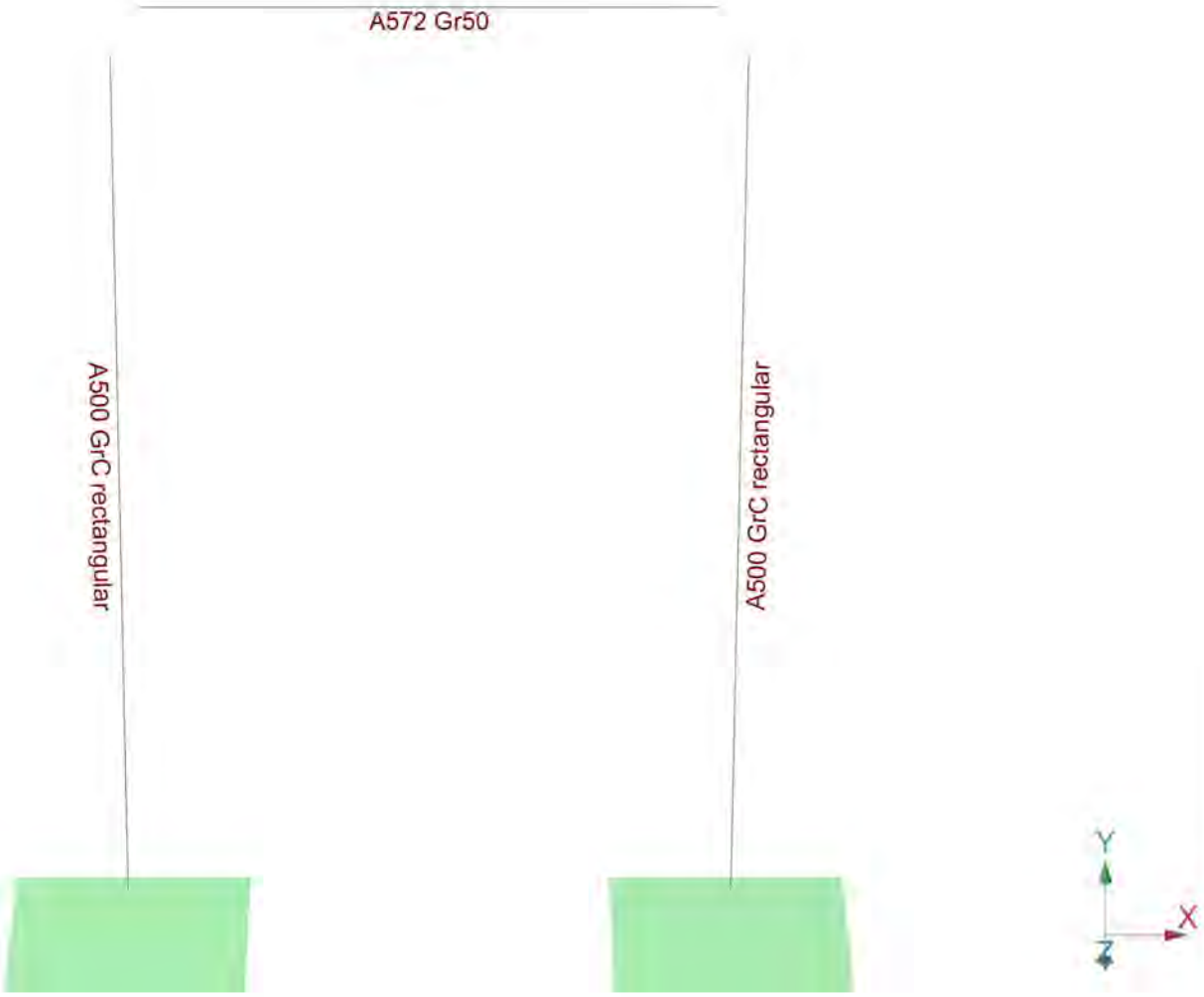
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
3D Rendering



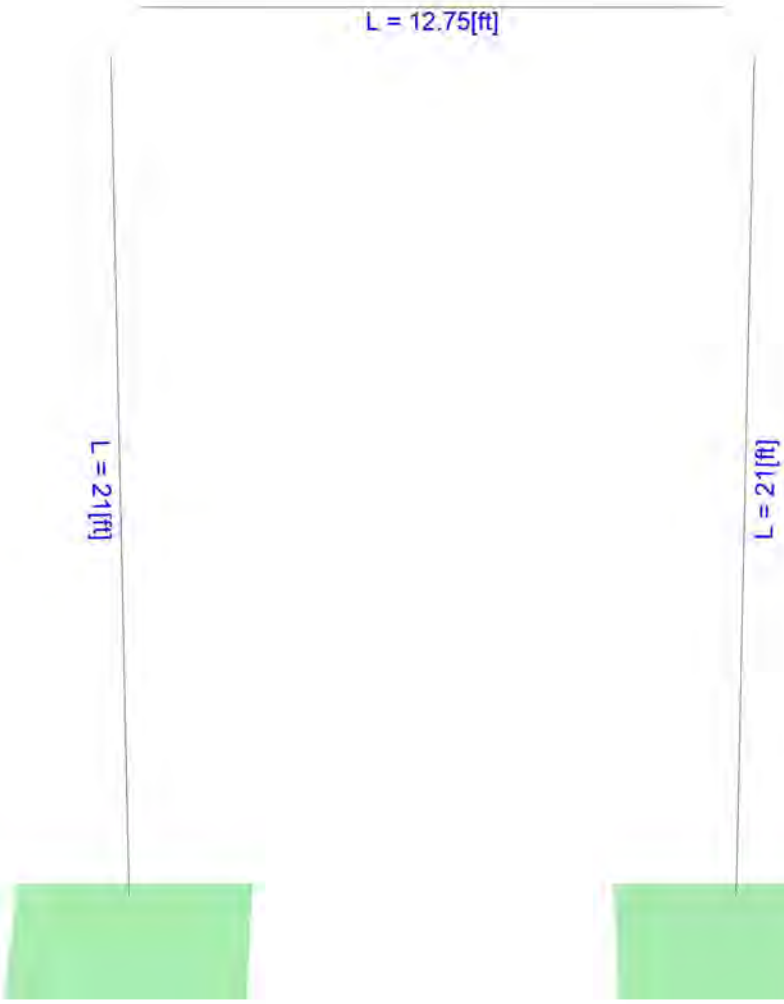
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms Member Sizes



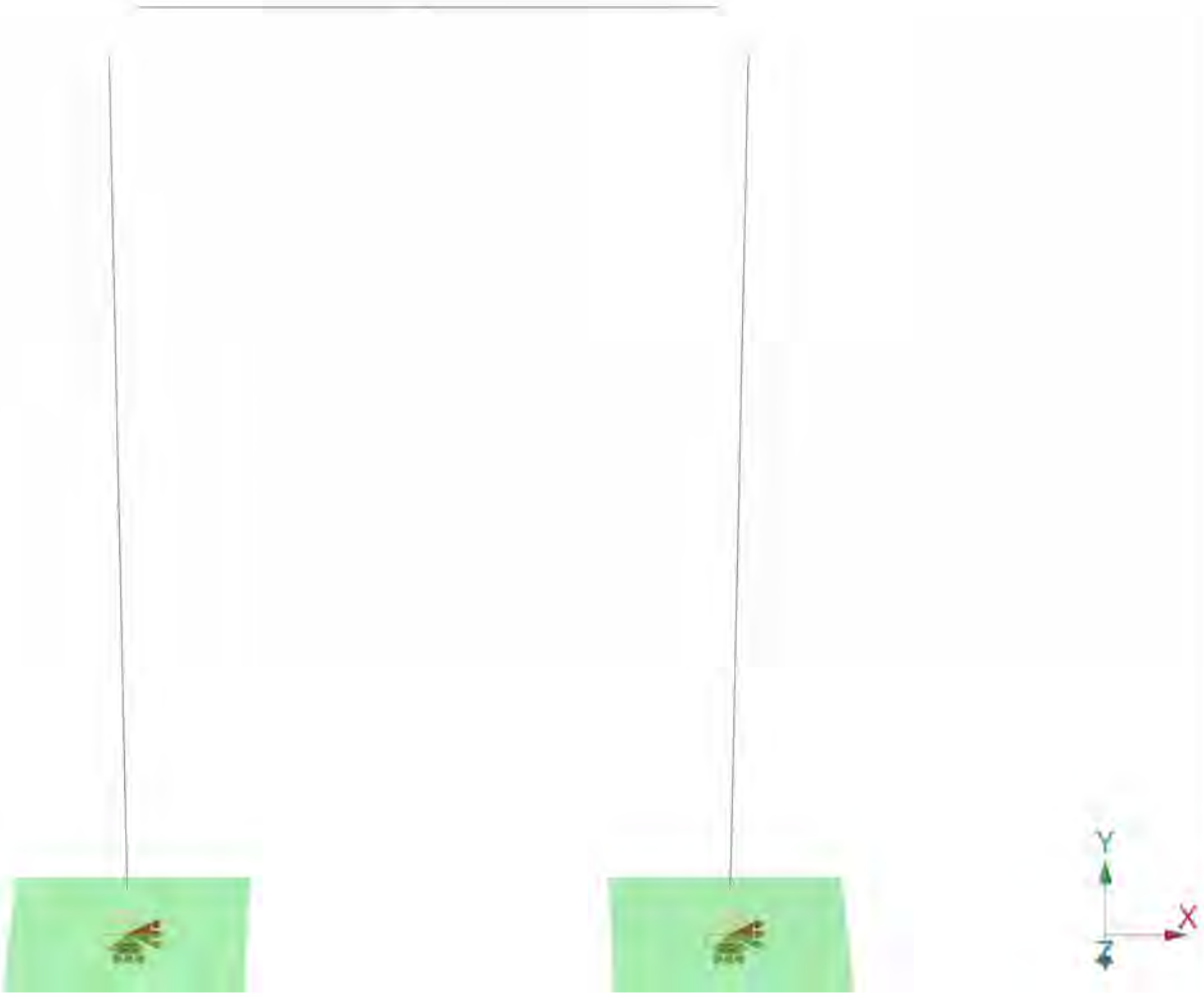
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
Member Material



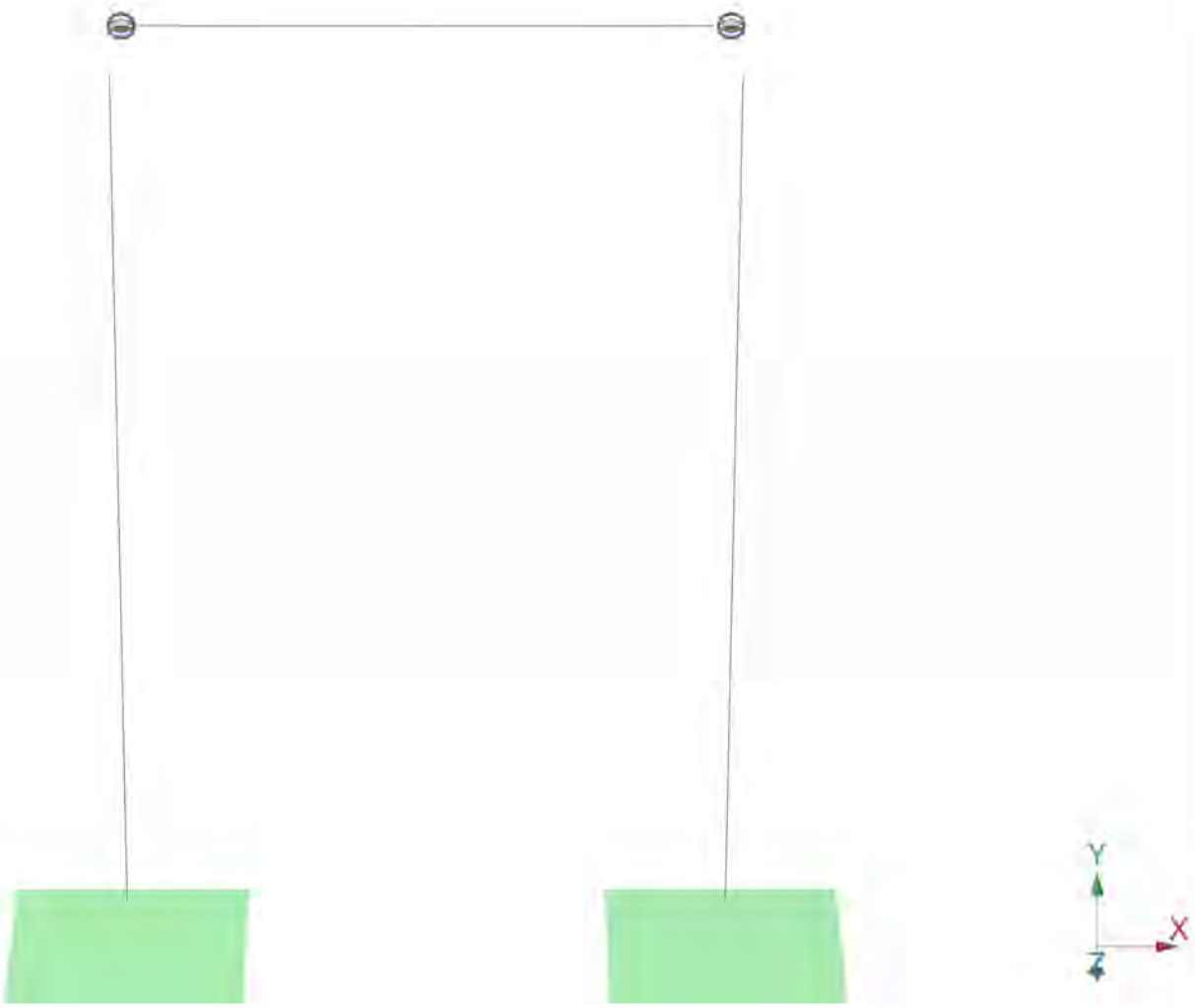
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms Member Lengths



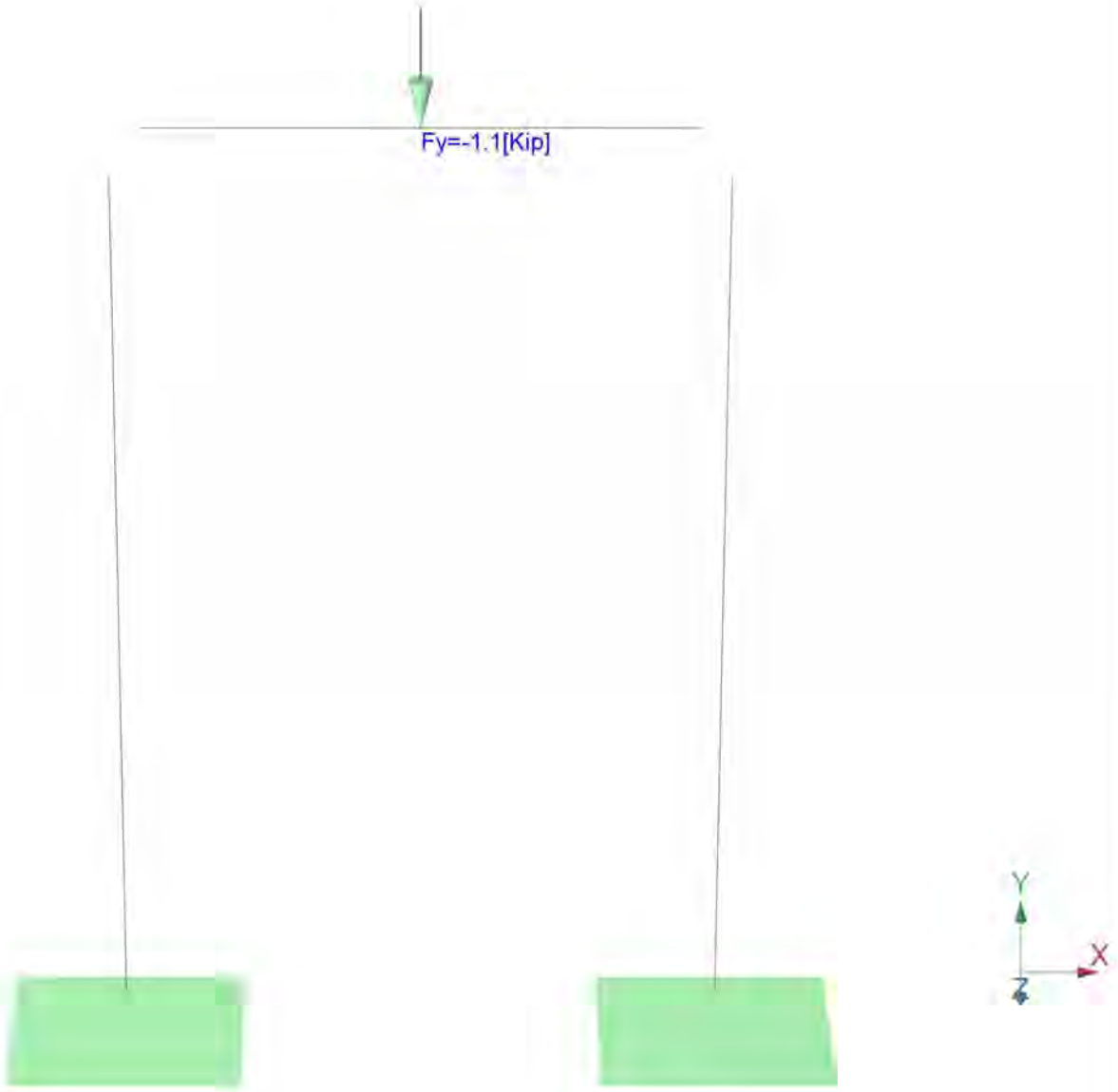
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
Column Supports



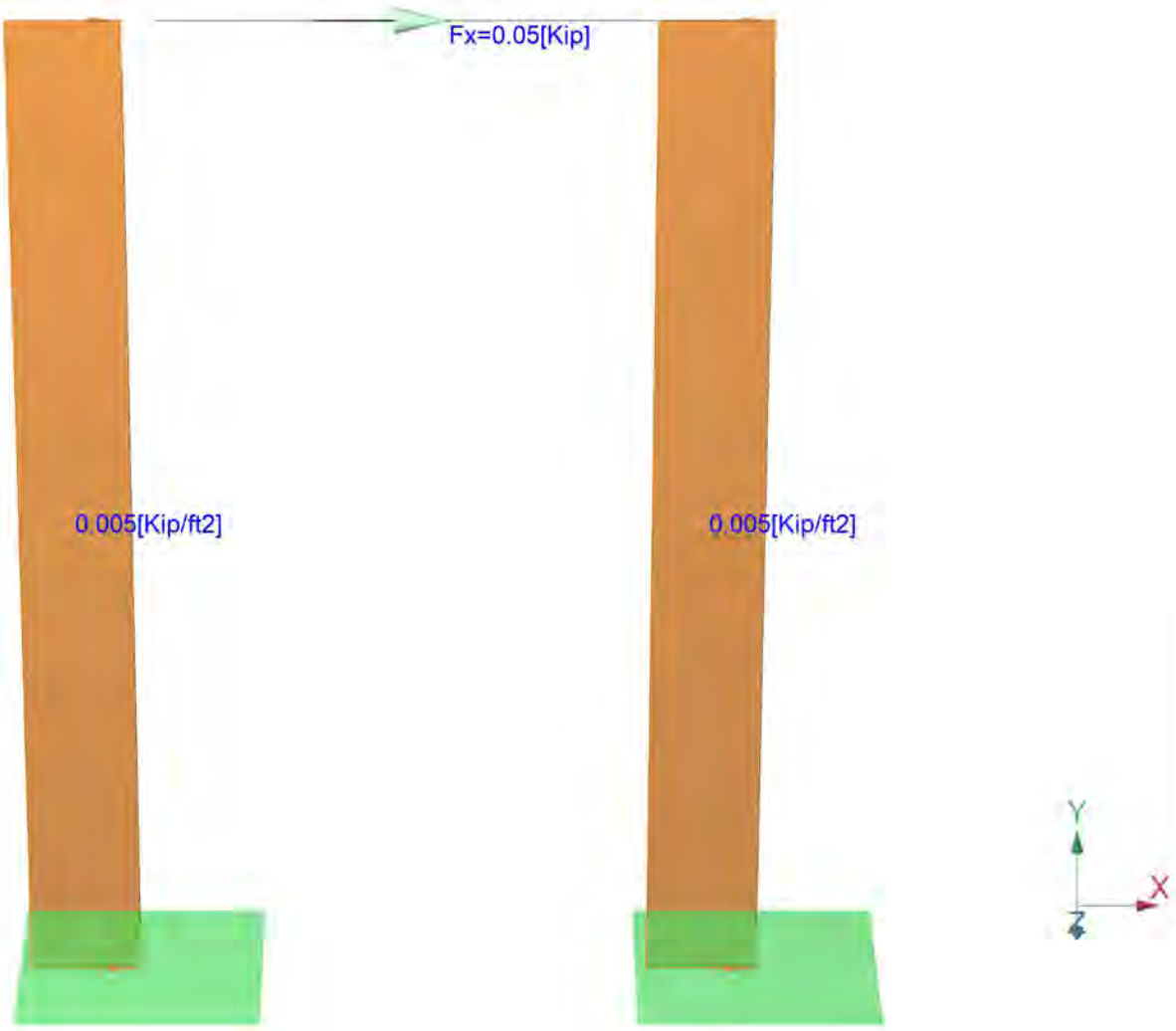
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
Beam Hinges



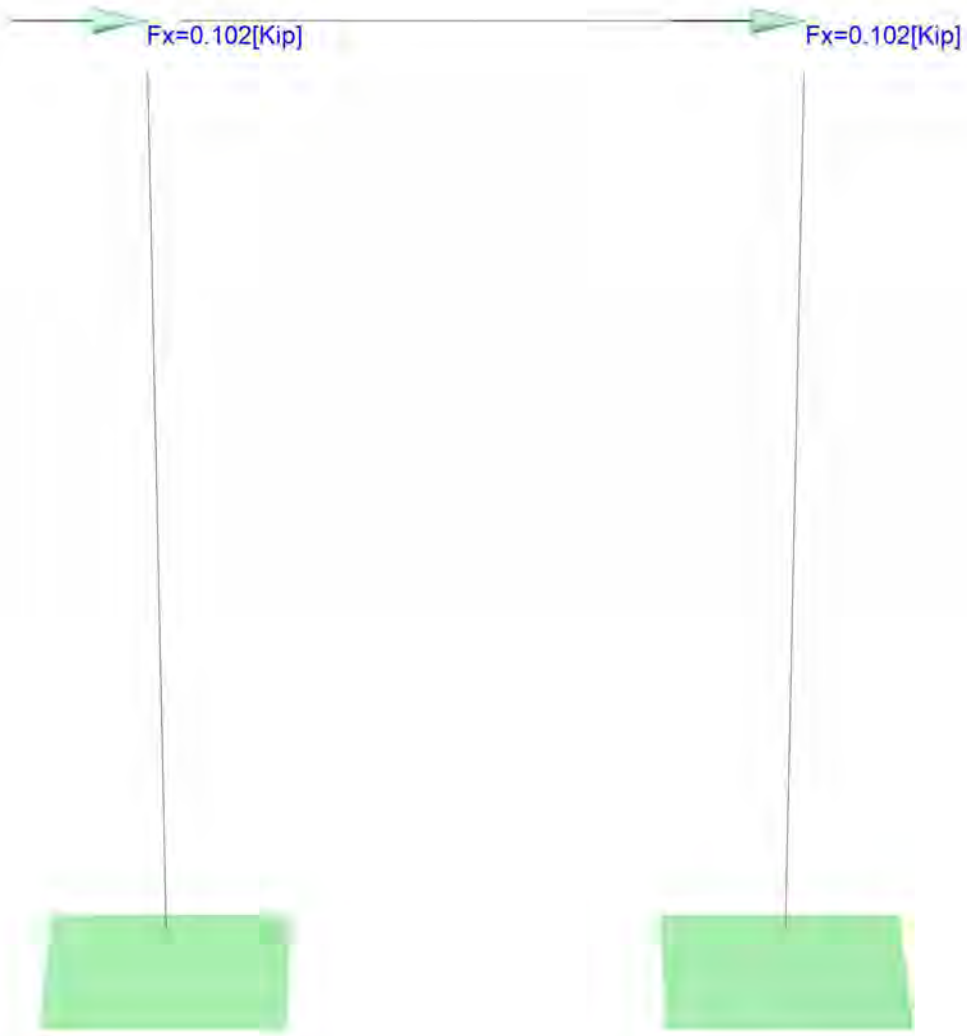
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
Dead Load



Cable Tray Supports in Hall Near Electrical and Mechanical Rooms
Internal Wind Load in X Direction

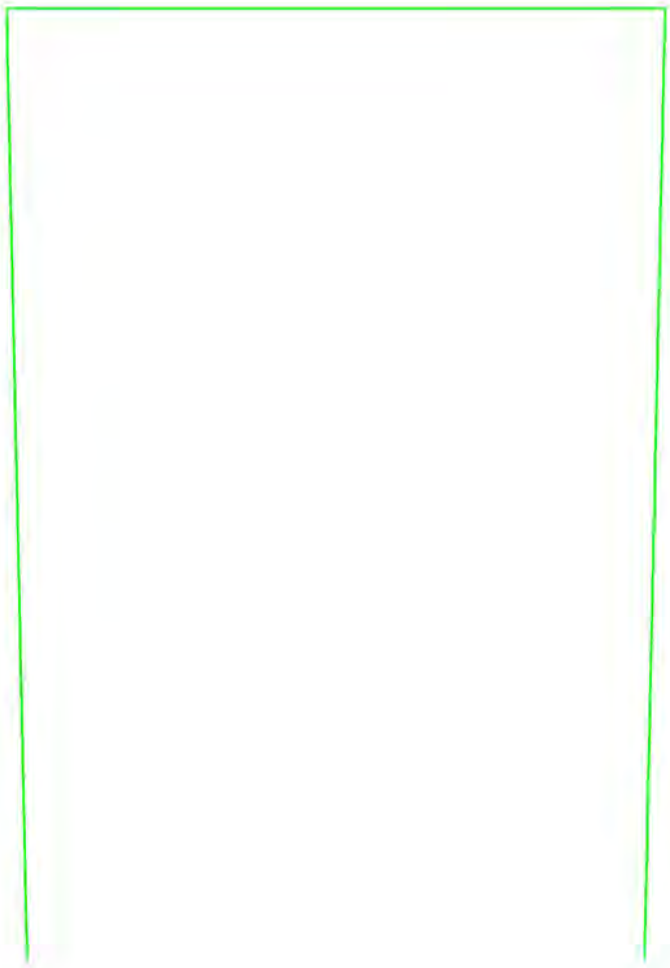


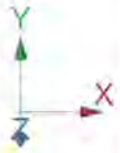
Cable Tray Supports in Hall Near Electrical and Mechanical Rooms Lateral Load in X Direction



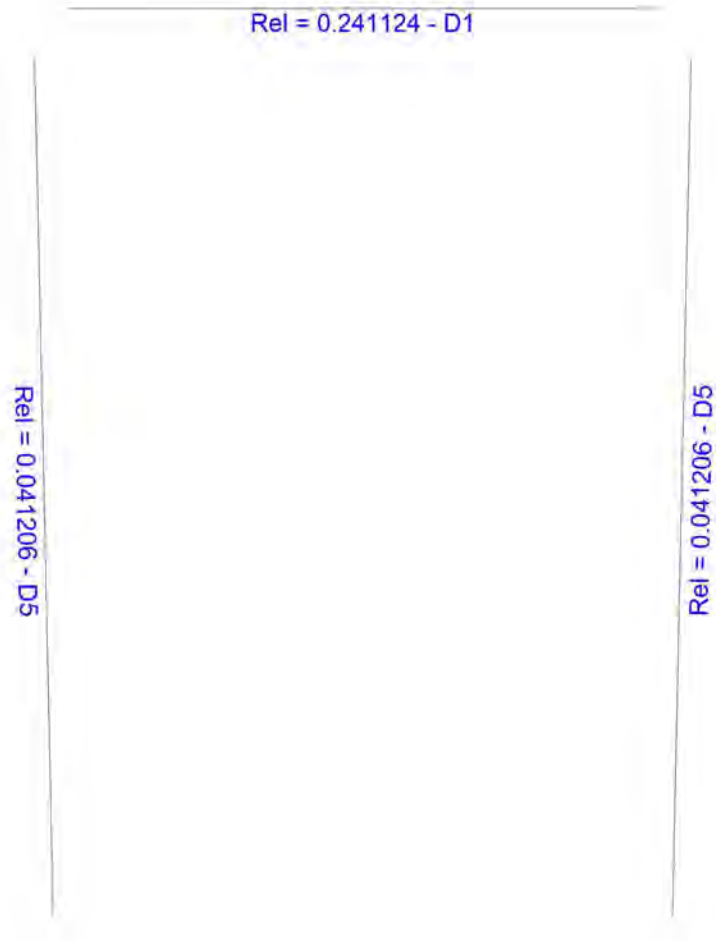
Design status
- Not designed
- Error on design
- Design O.K.
- With warnings

Cable Tray Supports in Hall Near Electrical and Mechanical Rooms Design Status





Cable Tray Supports in Hall Near Electrical and Mechanical Rooms Design Status Ratios



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 12:49 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pipe Supports BLDG 1\Interior\H-Frame - Cable Tray\h-fRAME - Interior.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5WLx
D3=1.2DL+WLx
D4=0.9DL+WLx
D5=1.2DL+EQx
D6=0.9DL+EQx

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	W 12X14	**3**	D1 at 50.00%	**0.24**	**OK**	
Post	***HSS_SQR 6X6X3_8***	**1**	D5 at 0.00%	**0.04**	**OK**	
		2	D5 at 0.00%	0.04	OK	

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Hallway Post - HSS6x6x3/8	Date:	8/17/2026
Fastening point:			

Specifier's comments:

1 Input data



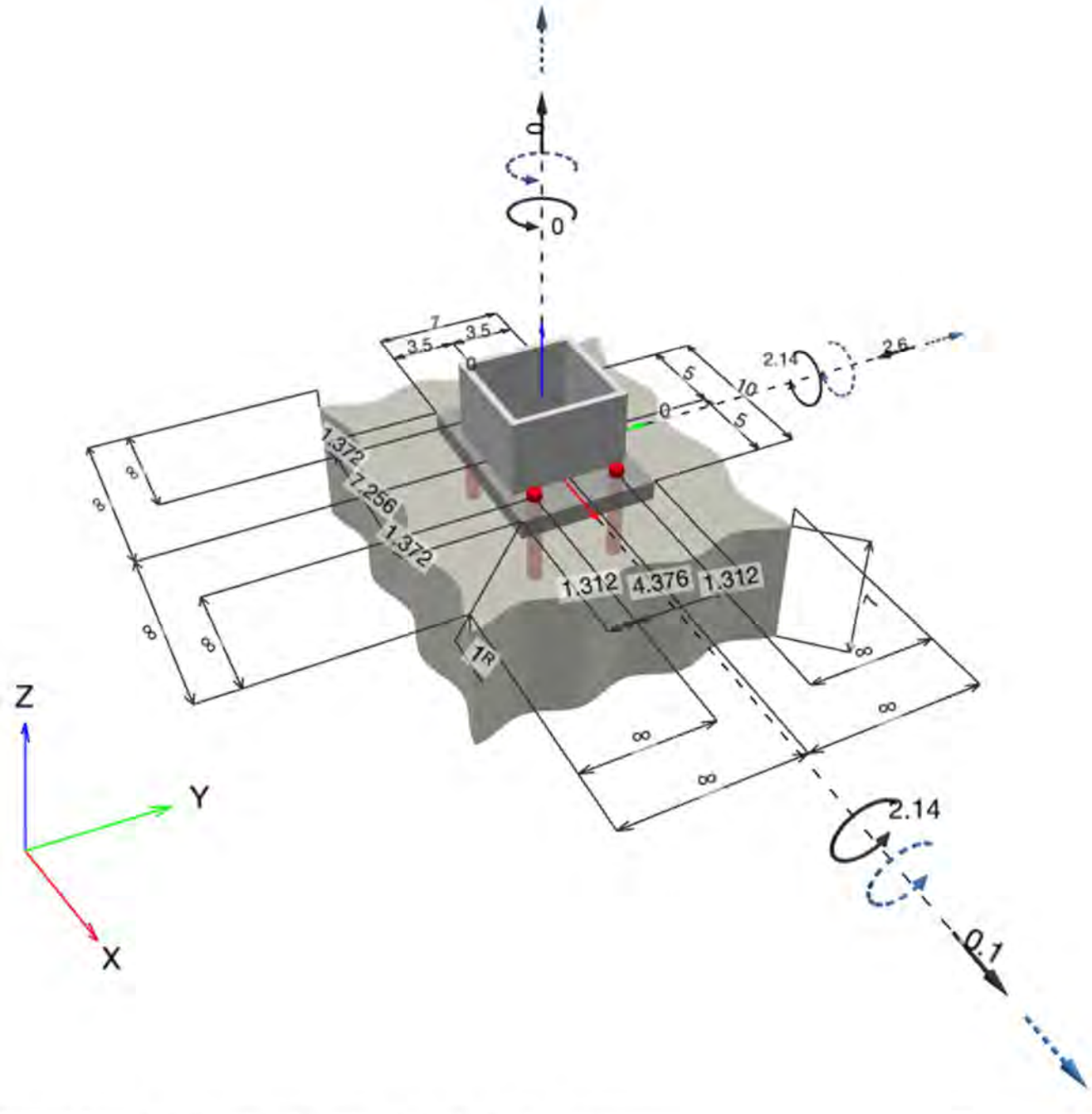
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418085 KH-EZ 3/4"x7"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	3/1/2026 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R]:	l_x x l_y x t = 10.000 in. x 7.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS6X6X.375; (L x W x T) = 6.000 in. x 6.000 in. x 0.375 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 7.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Hallway Post - HSS6x6x3/8	Date:	8/17/2026
Fastening point:			

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
PROFIS Engineering (c) 2003-2026 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Hallway Post - HSS6x6x3/8	Date:	8/17/2026
Fastening point:			

1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	-	-3.000	-	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	-	-	-	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	-	-	-	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	-	0.100	0.100	0.65000	-	-
E (Earthquake)	-	-	0.100	0.100	-	2.14000	2.14000	-

1.2 Load combination and design results

1.2.1 Load combination

Load case	Load combination
Equation (16-1)	1.4 (D + F)
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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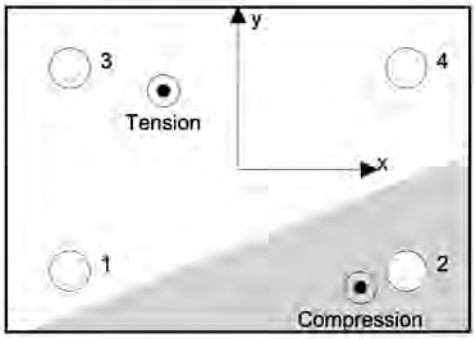
1.2.2 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
Equation (16-1)	1.4 (D + F)	N = 0.000; V_x = 0.000; V_y = -4.200; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	20
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L	N = 0.000; V_x = 0.000; V_y = -3.600; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)	N = 0.050; V_x = 0.000; V_y = -3.550; M_x = 0.32500; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)	N = 0.050; V_x = 0.000; V_y = -3.550; M_x = 0.32500; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)	N = 0.050; V_x = 0.000; V_y = -3.550; M_x = 0.32500; M_y = 0.00000; M_z = 0.00000;	no	17
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)	N = 0.100; V_x = 0.000; V_y = -3.500; M_x = 0.65000; M_y = 0.00000; M_z = 0.00000;	no	23
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)	N = 0.100; V_x = 0.000; V_y = -3.500; M_x = 0.65000; M_y = 0.00000; M_z = 0.00000;	no	23
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)	N = 0.100; V_x = 0.000; V_y = -3.500; M_x = 0.65000; M_y = 0.00000; M_z = 0.00000;	no	23
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S	N = 0.000; V_x = 0.100; V_y = -3.500; M_x = 2.14000; M_y = 2.14000; M_z = 0.00000;	no	85
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)	N = 0.100; V_x = 0.000; V_y = -2.600; M_x = 0.65000; M_y = 0.00000; M_z = 0.00000;	no	23
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)	N = 0.000; V_x = 0.100; V_y = -2.600; M_x = 2.14000; M_y = 2.14000; M_z = 0.00000;	no	85

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2 Load case/Resulting anchor forces

Controlling load case: Equation (16-7) 0.9 (D + F) + 1.0 (E) + 1.6 (H)



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.673	0.650	0.025	-0.650
2	0.000	0.650	0.025	-0.650
3	3.694	0.650	0.025	-0.650
4	1.678	0.650	0.025	-0.650

Max. concrete compressive strain: 0.23 [‰]
Max. concrete compressive stress: 1,006 [psi]
Resulting tension force in (x/y)=(-1.614/1.701): 6.045 [kip]
Resulting compression force in (x/y)=(2.634/-2.548): 6.045 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	3.694	20.808	18	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	6.045	7.158	85	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
32.013	0.650	20.808	3.694

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.770	0.405	0.971	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.690	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
259.47	127.92	0.933	0.853	1.000	1.000	6.816

Results

N_{cbg} [kip]	$\phi_{concrete}$	ϕN_{cbg} [kip]	N_{ua} [kip]
11.012	0.650	7.158	6.045

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.650	9.996	7	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2.602	21.725	12	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [kip]
16.660

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.660	0.600	9.996	0.650

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.690	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
291.23	127.92	1.000	1.000	1.000	1.000	6.816

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
31.035	0.700	21.725	2.602

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.845	0.120	5/3	79	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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7 Installation data

Profile: Square HSS (AISC), HSS6X6X.375; (L x W x T) = 6.000 in. x 6.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)

Item number: 418085 KH-EZ 3/4"x7"

Maximum installation torque: 0.09501 ft.kip

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.375 in.

Minimum thickness of the base material: 7.000 in.

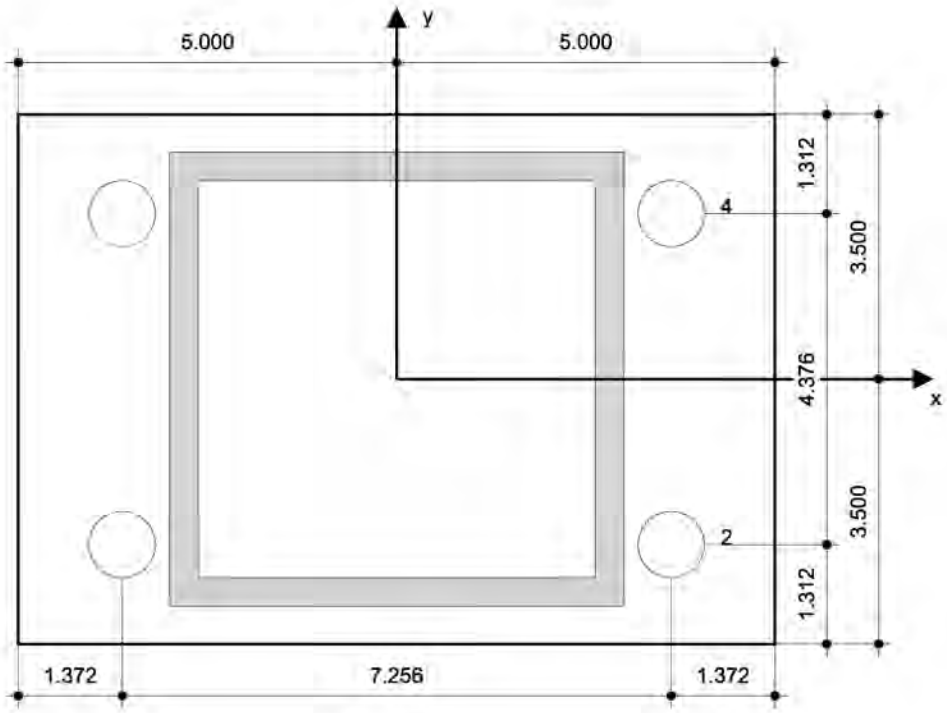
Hilti ∅ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 5 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hilti SIW 9-A22 Impact Wrench

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Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.628	-2.188	-	-	-	-
2	3.628	-2.188	-	-	-	-
3	-3.628	2.188	-	-	-	-
4	3.628	2.188	-	-	-	-

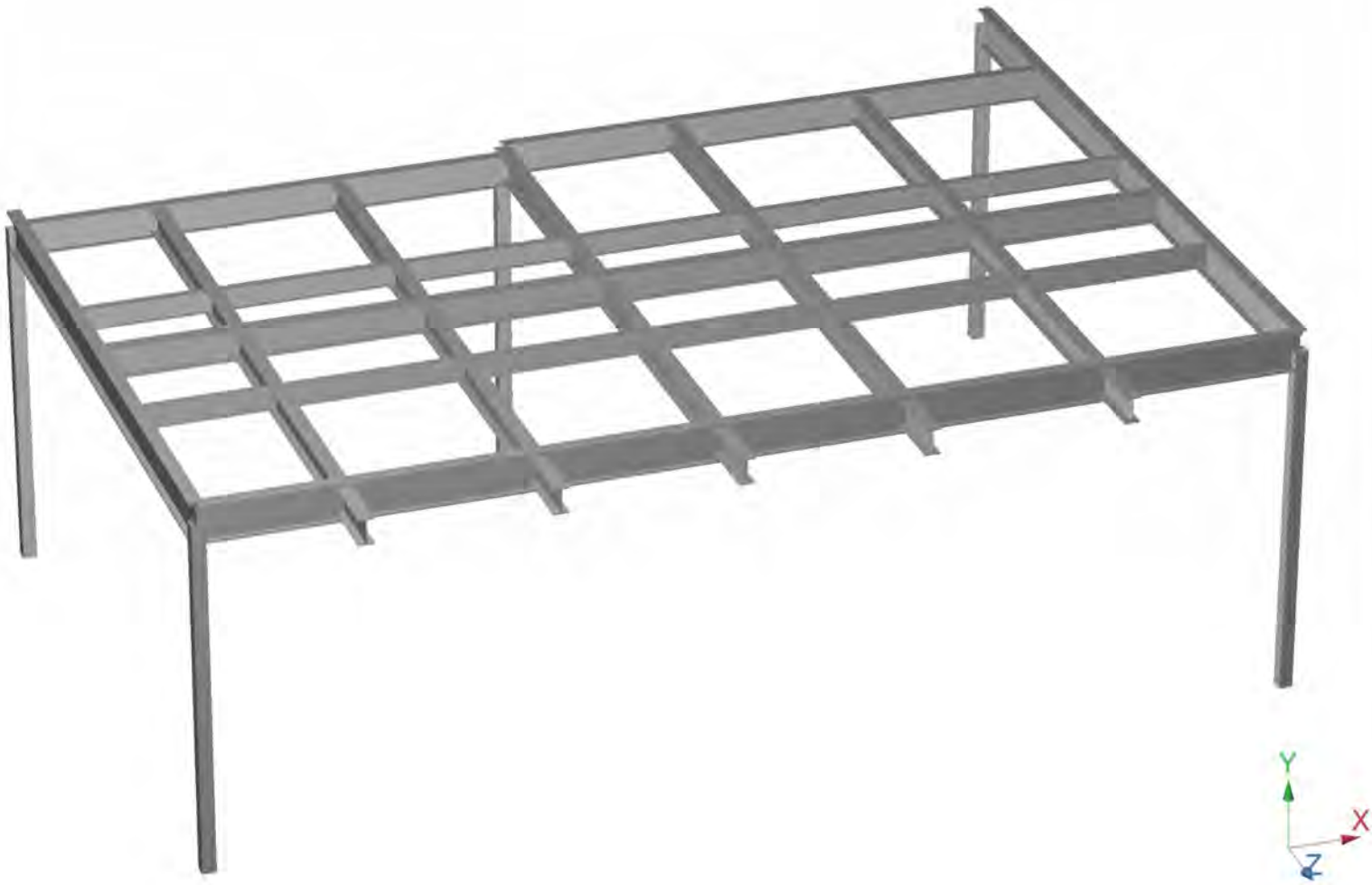
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8 Remarks; Your Cooperation Duties

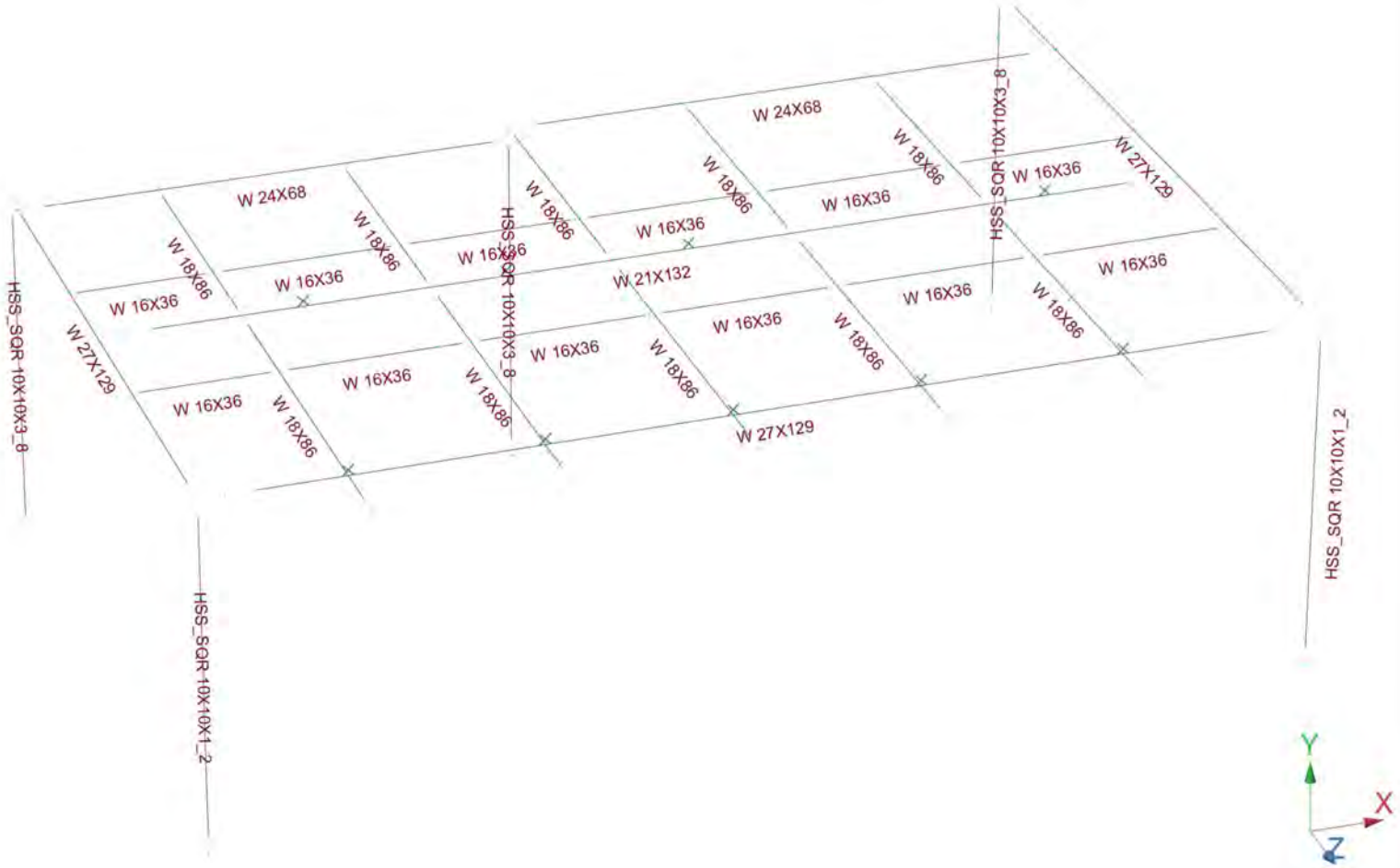
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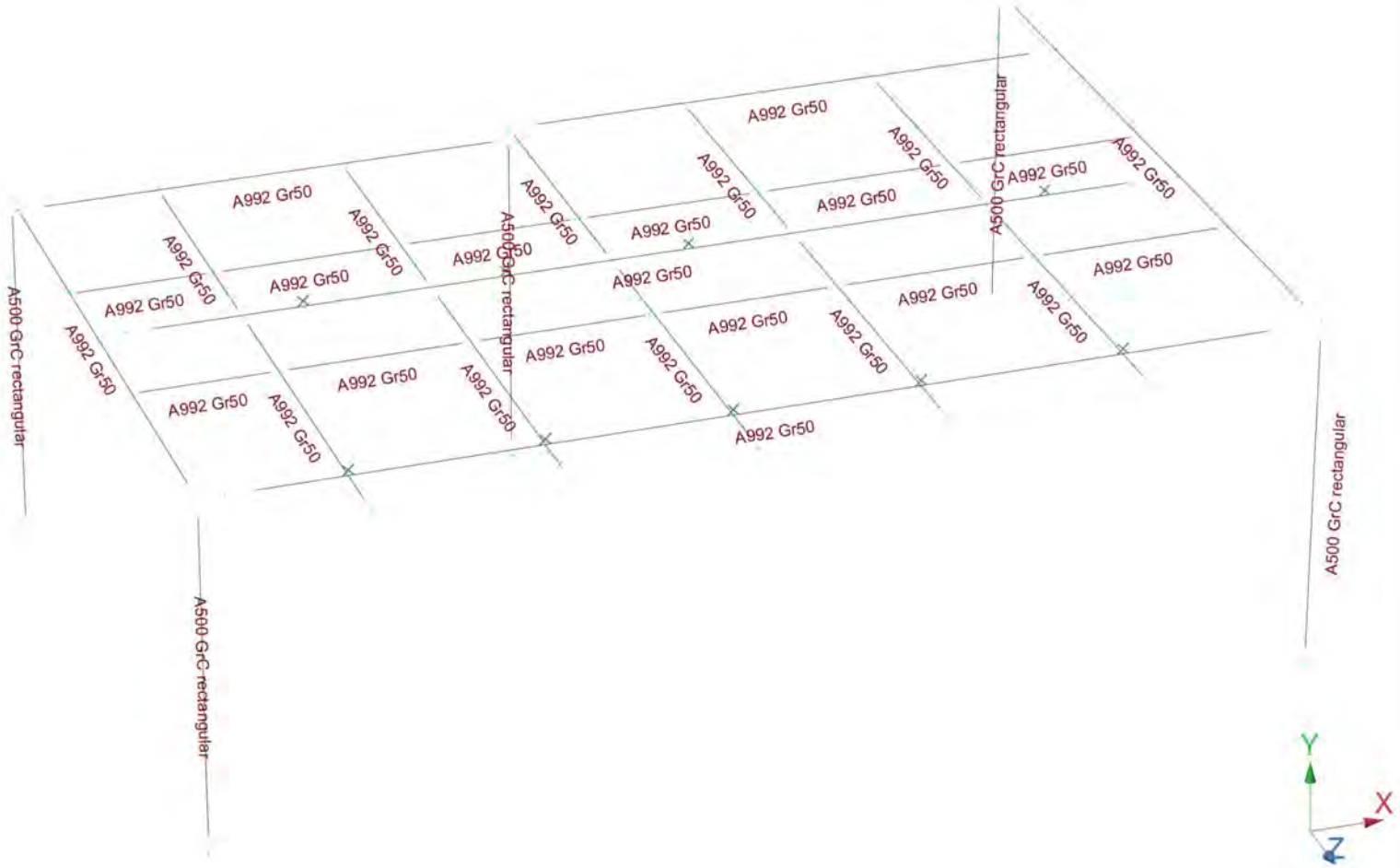
Equipment Supports in Mechanical Room
3D Rendering



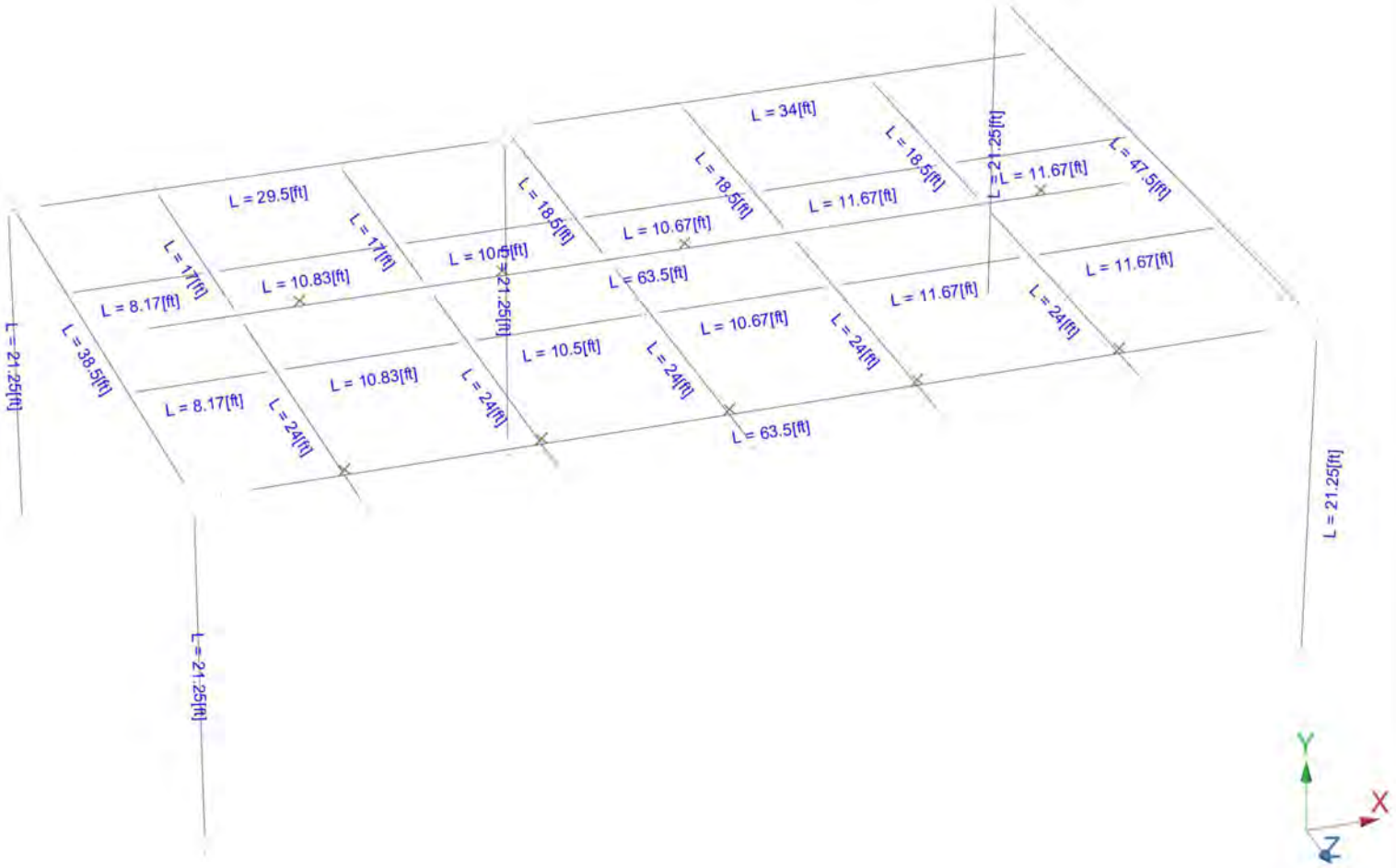
Equipment Supports in Mechanical Room
Member Sizes



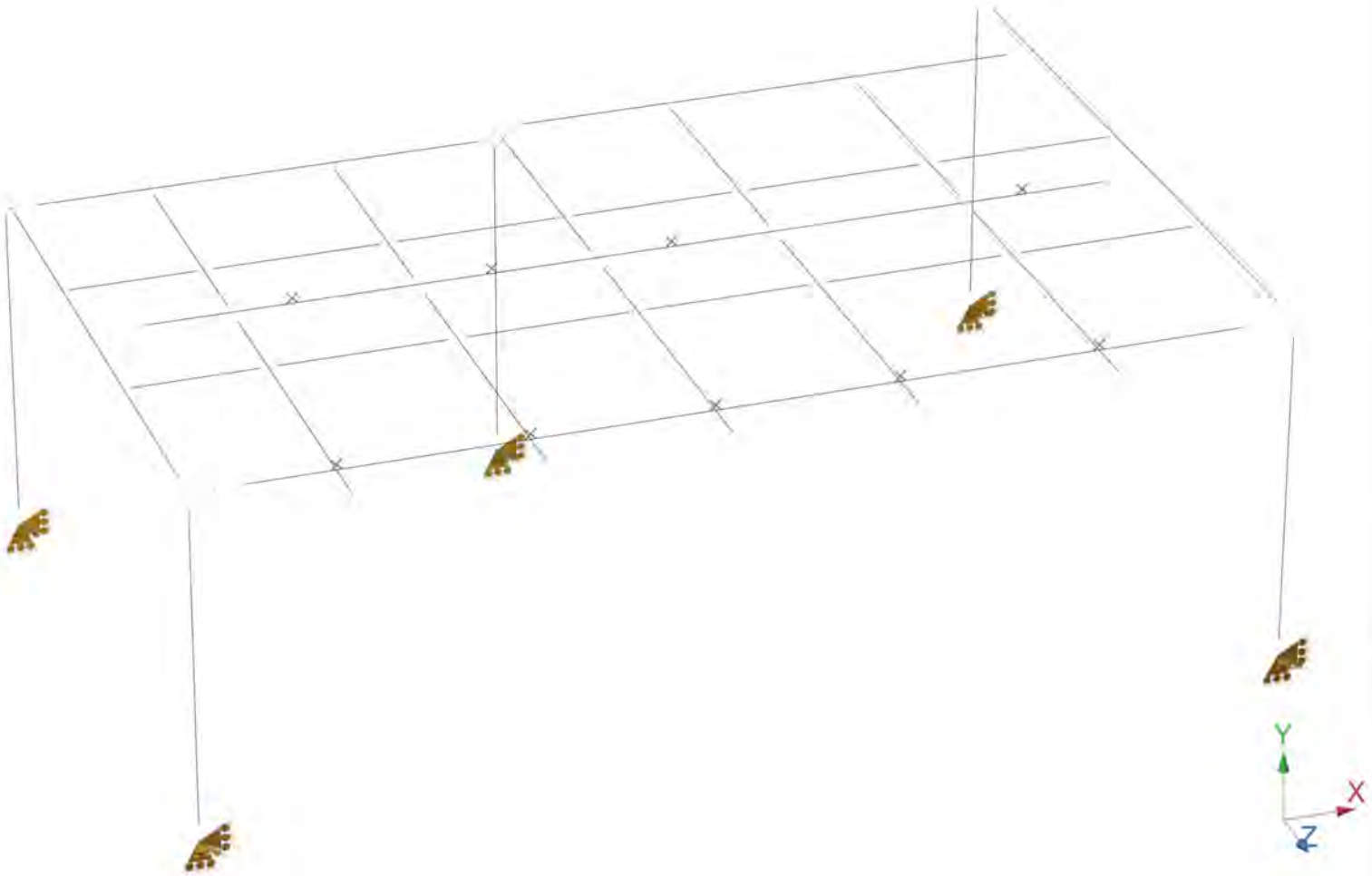
Equipment Supports in Mechanical Room
Member Material



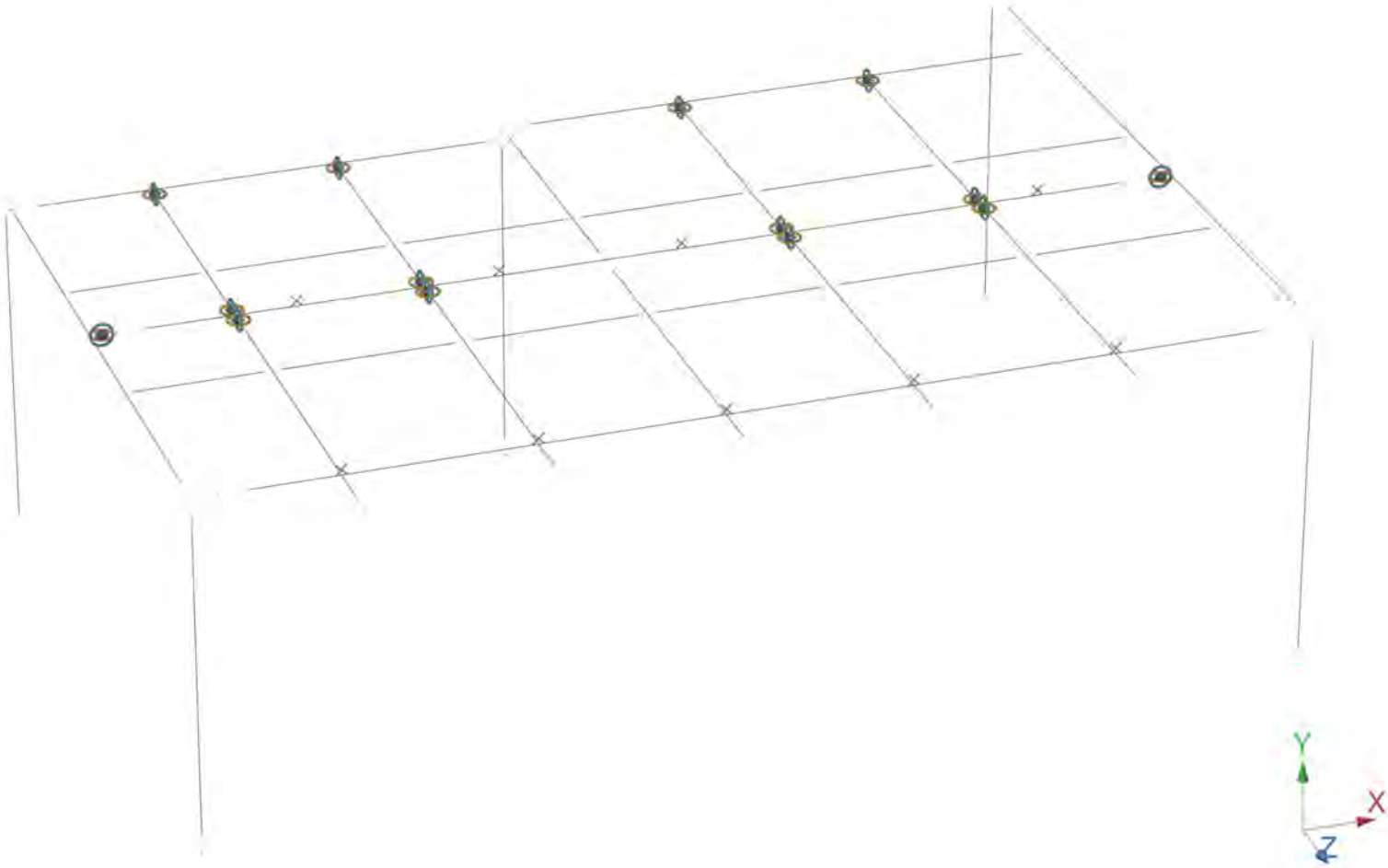
Equipment Supports in Mechanical Room
Member Lengths



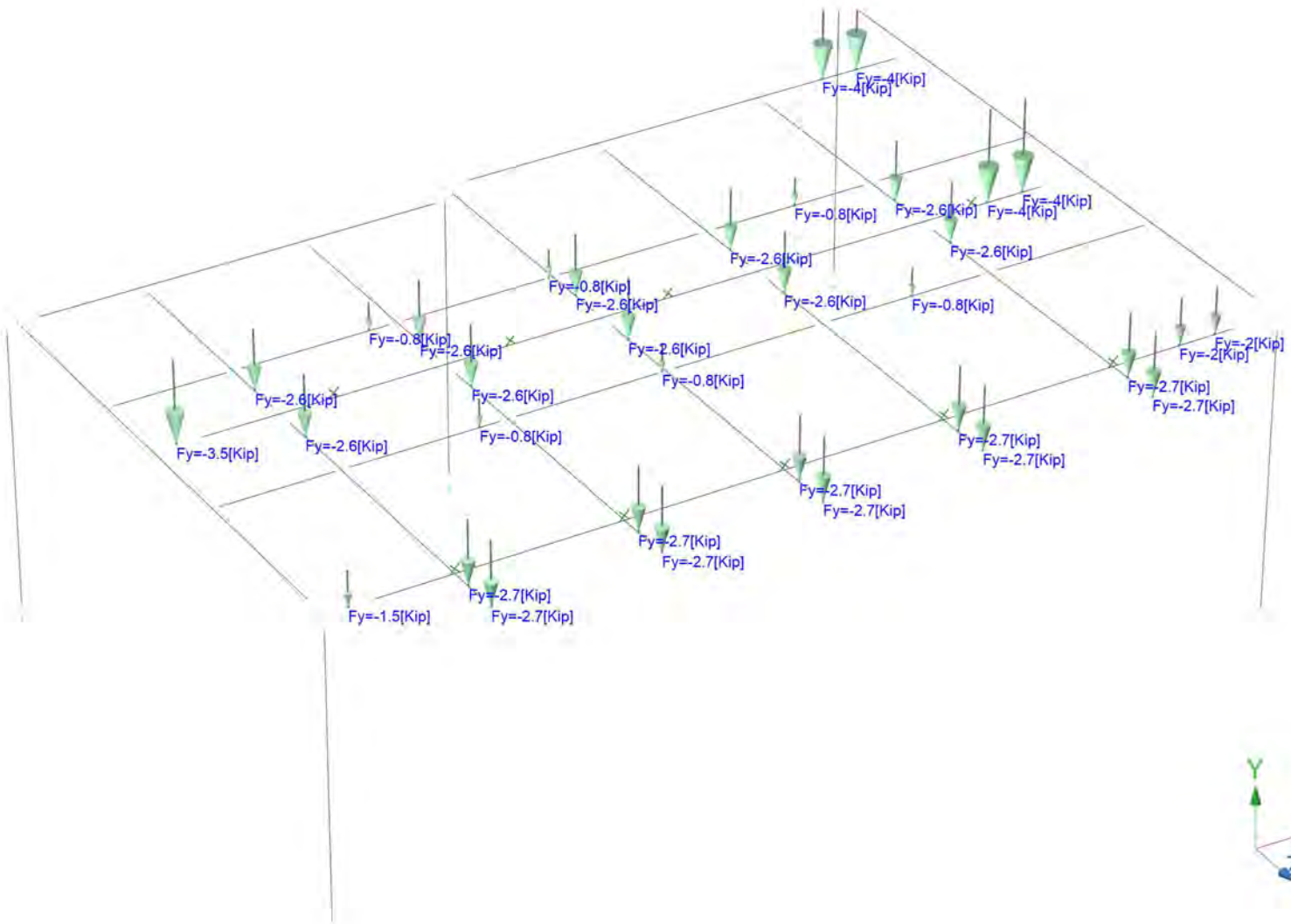
Equipment Supports in Mechanical Room
Column Supports



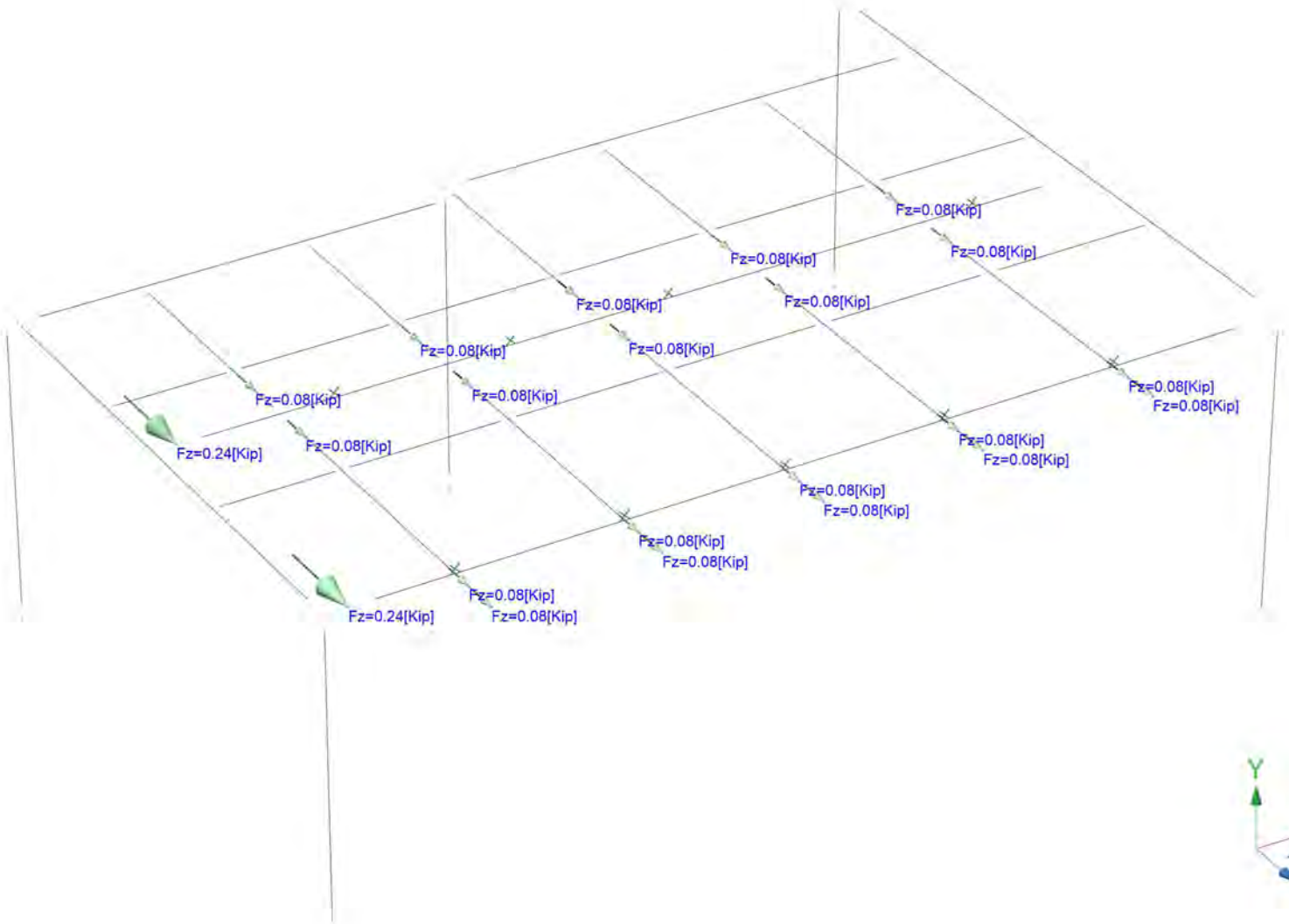
Equipment Supports in Mechanical Room
Beam Hinges



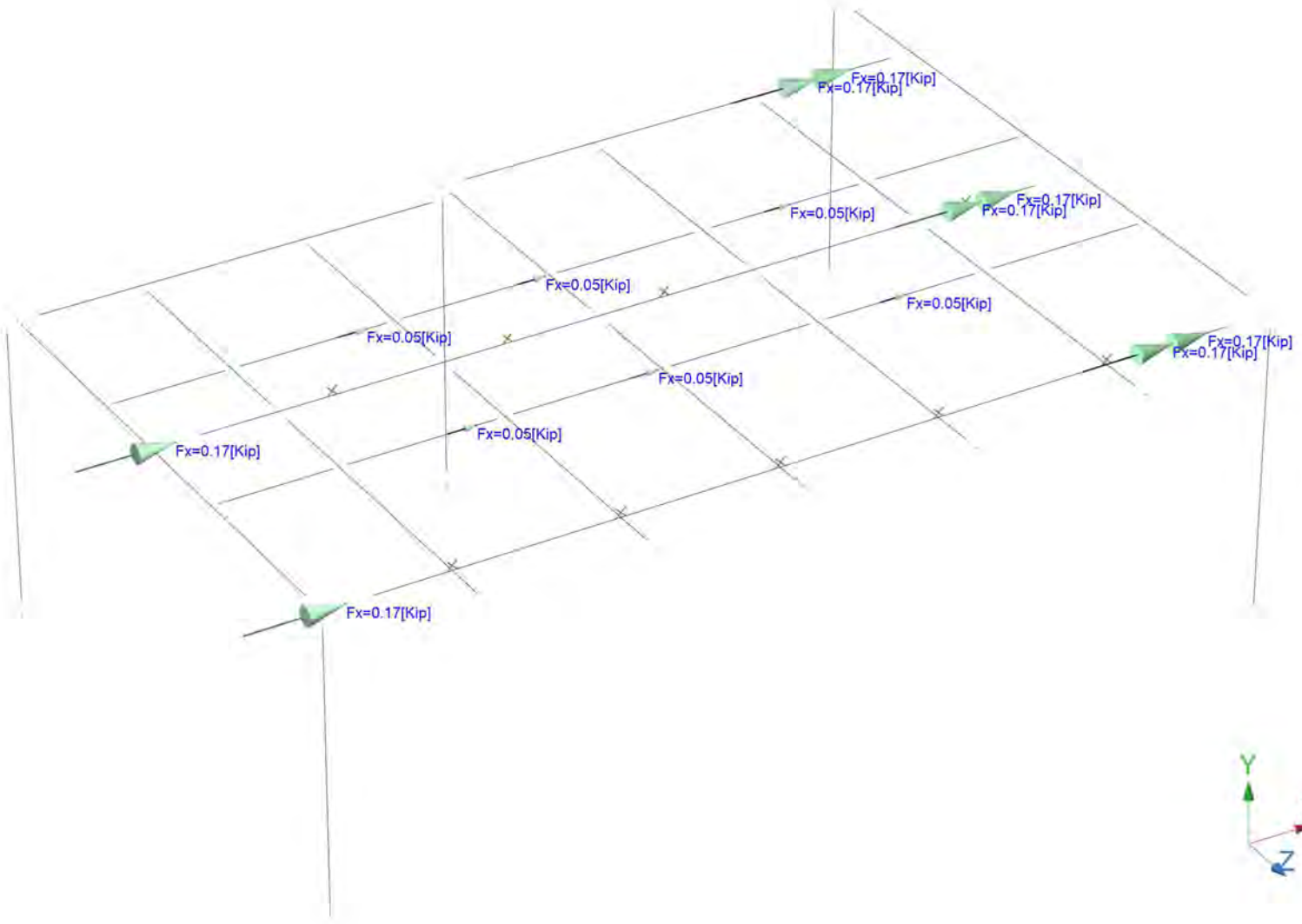
Equipment Supports in Mechanical Room
Dead Loads



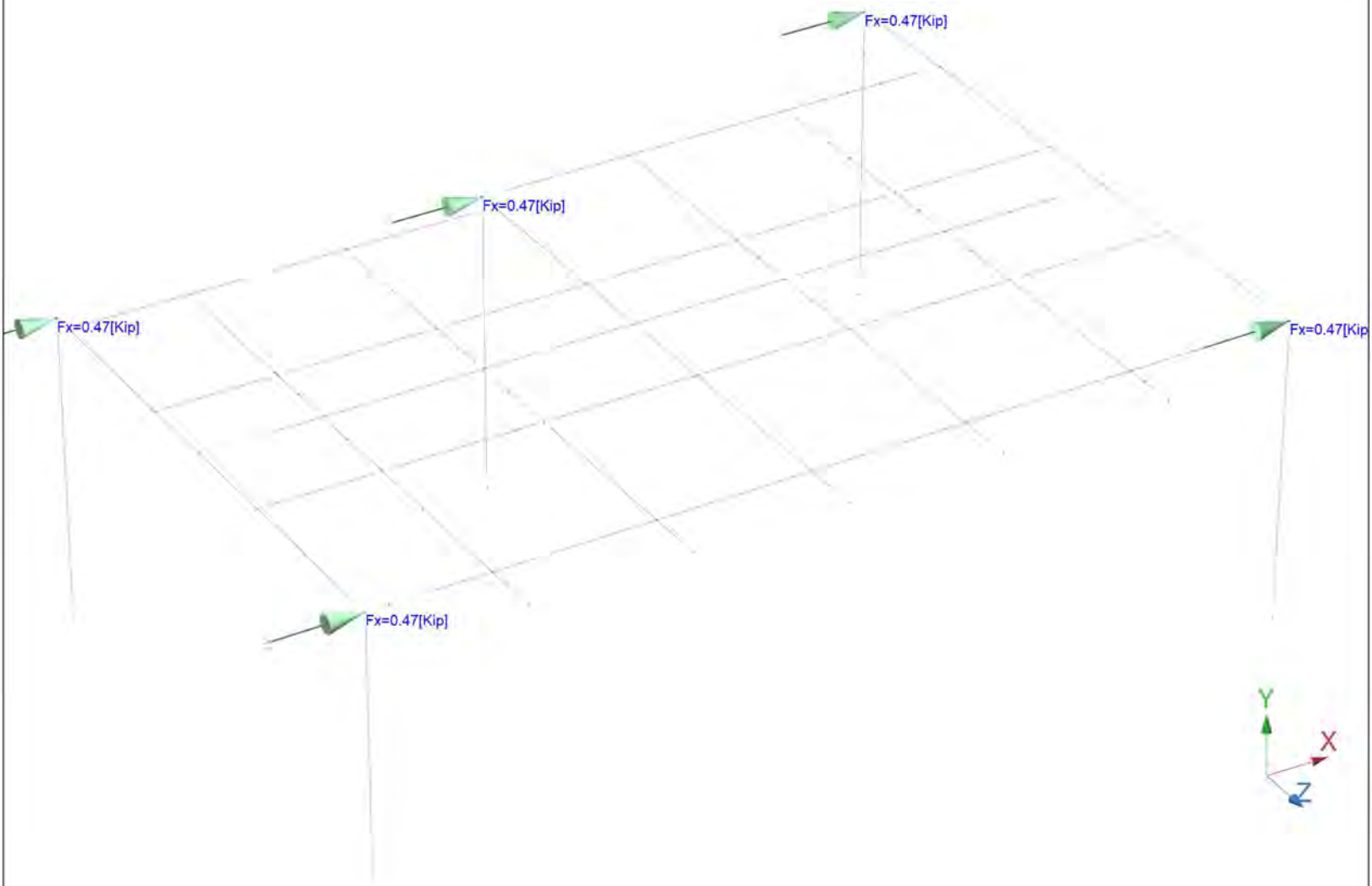
Equipment Supports in Mechanical Room
Internal Wind Pressure in Z Direction



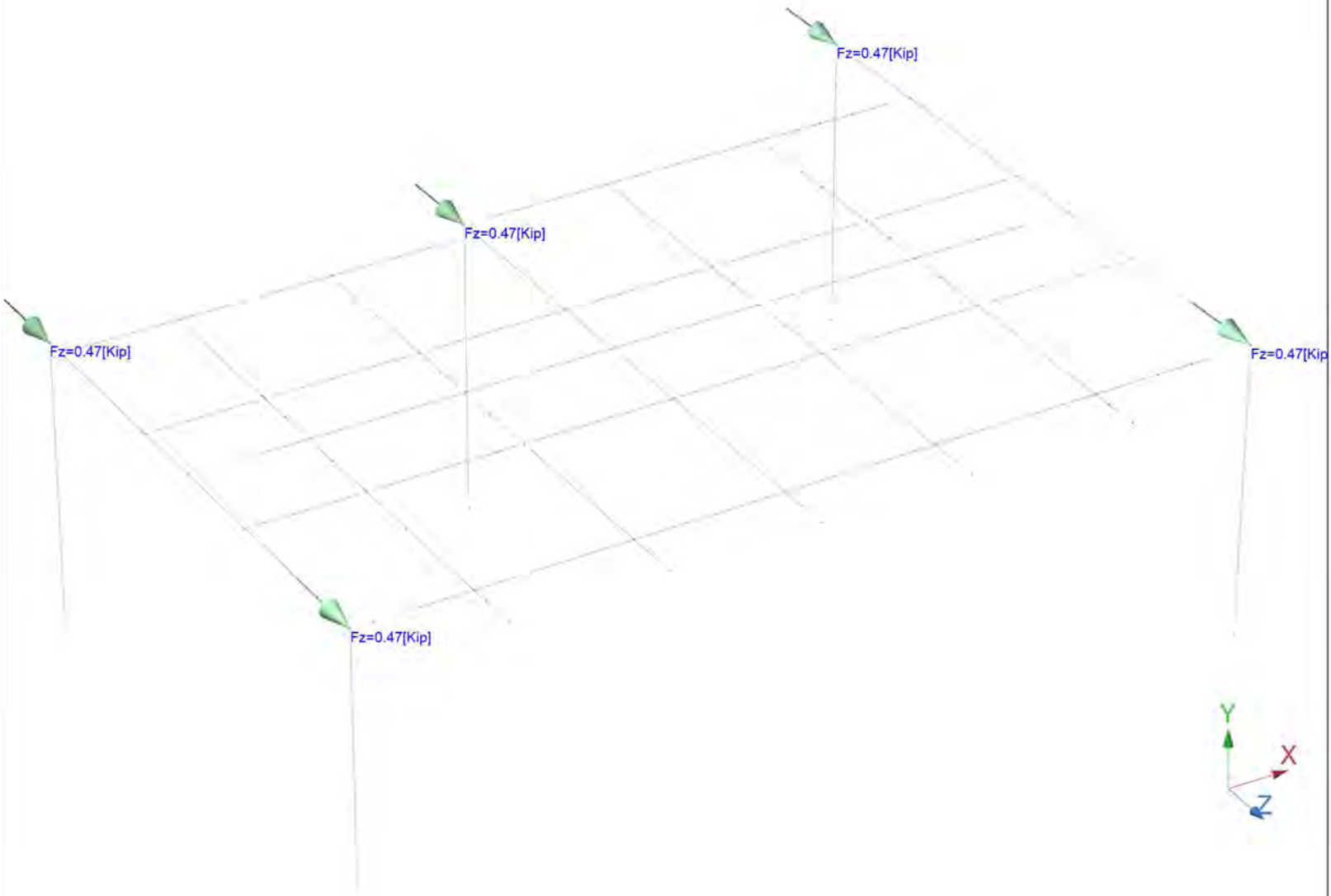
Equipment Supports in Mechanical Room
Internal Wind Pressure in X Direction



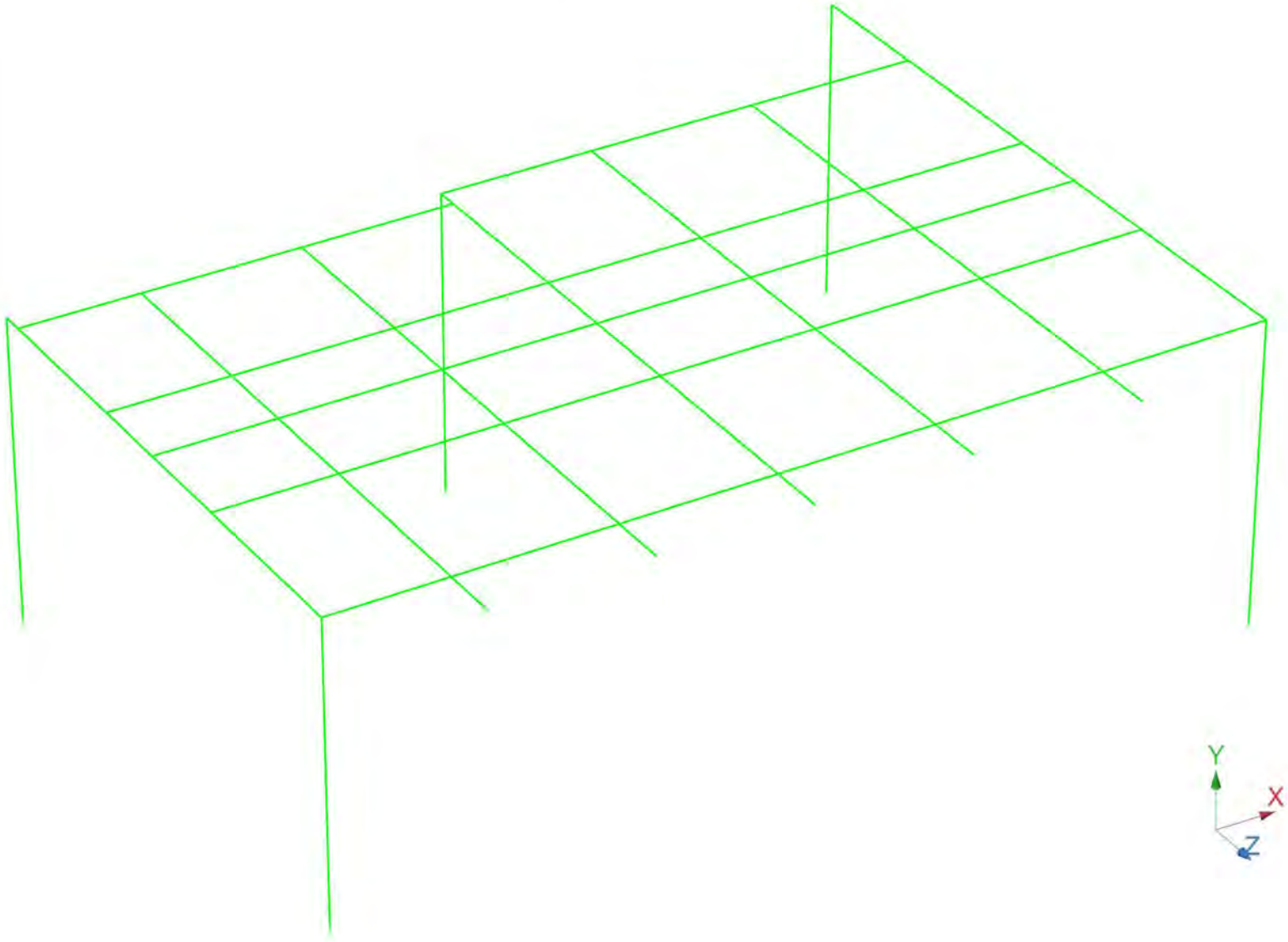
Equipment Supports in Mechanical Room
Lateral Load in X Direction



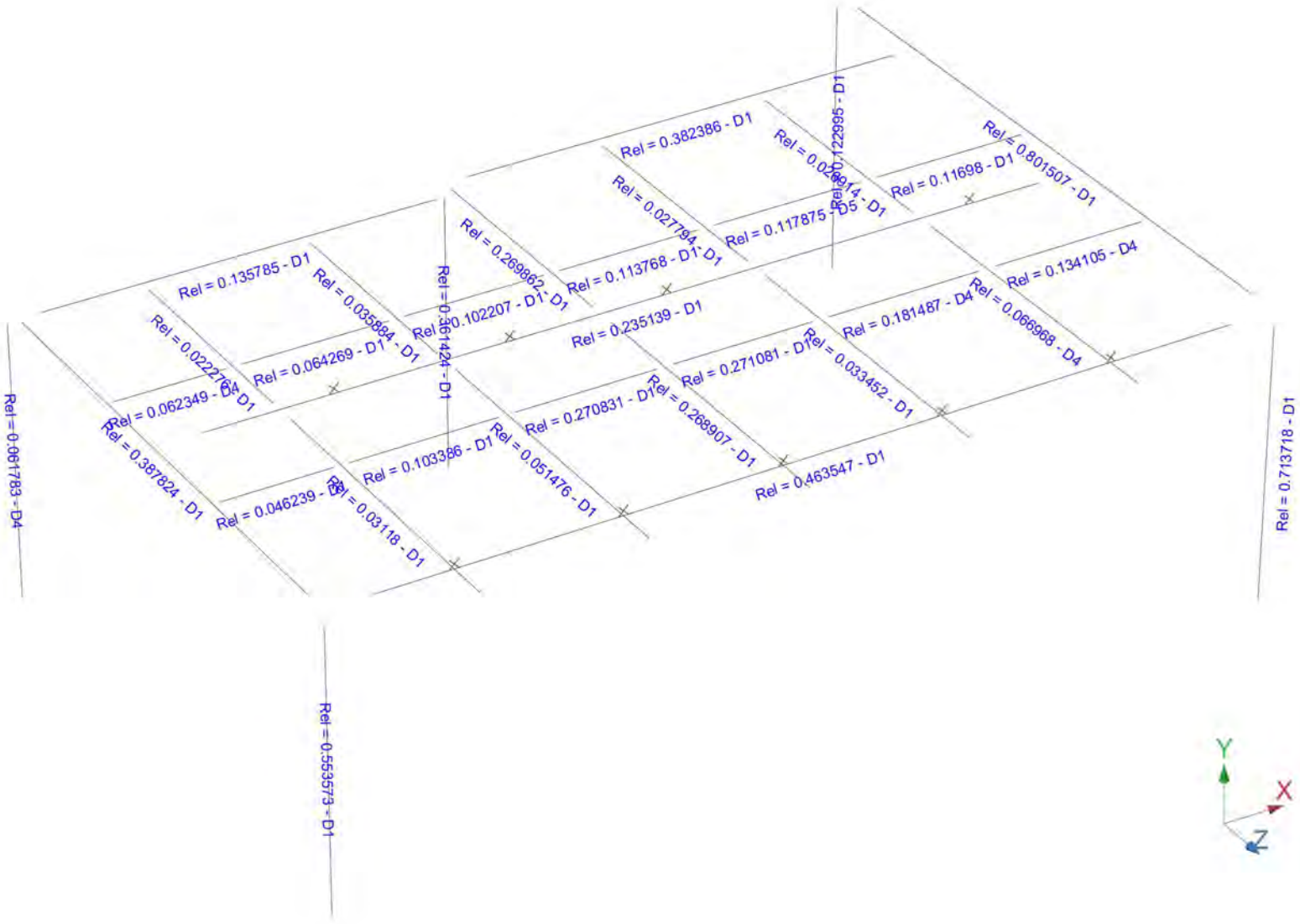
Equipment Supports in Mechanical Room
Lateral Load in Z Direction



Equipment Supports in Mechanical Room
Design Status



Equipment Supports in Mechanical Room
Design Status Ratios



Current Date: 8/17/2026 11:59 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pipe Supports BLDG 1\Interior\Mechanical Room\Piping Support.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5WLz
D3=1.2DL+0.5WLx
D4=1.2DL+WLz
D5=1.2DL+WLx
D6=0.9DL+WLz
D7=0.9DL+WLx

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	W 16X36	**40**	D1 at 100.00%	0.05	OK	
		42	D1 at 82.69%	0.10	OK	
		47	D1 at 100.00%	0.27	OK	
		48	D1 at 0.00%	**0.27**	**OK**	
		49	D4 at 0.00%	0.18	OK	
		50	D4 at 0.00%	0.13	OK	
		51	D4 at 100.00%	0.06	OK	
		53	D1 at 82.69%	0.06	OK	
		58	D1 at 97.62%	0.10	OK	
		59	D1 at 0.00%	0.11	OK	
		60	D5 at 0.00%	0.12	OK	
		61	D1 at 0.00%	0.12	OK	
Beam	***W 18X86***	**12**	D1 at 100.00%	0.27	OK	
		26	D1 at 16.67%	0.03	OK	
		27	D1 at 16.67%	0.05	OK	
		28	D1 at 70.14%	0.03	OK	
		29	D4 at 16.67%	0.07	OK	
		52	D1 at 33.33%	0.02	OK	
		54	D1 at 33.33%	0.04	OK	
		55	D1 at 0.00%	**0.27**	**OK**	
		56	D1 at 30.63%	0.03	OK	
		57	D1 at 30.63%	0.03	OK	
	W 27X129	**11**	D1 at 51.95%	0.39	OK	
		14	D1 at 42.11%	**0.80**	**OK**	
Girder	***W 21X132***	**17**	D1 at 36.75%	**0.24**	**OK**	
	W 24X68	**15**	D1 at 34.31%	**0.38**	**OK**	
		16	D1 at 64.41%	0.14	OK	
	W 27X129	**18**	D1 at 46.46%	**0.46**	**OK**	
Post	***HSS_SQR 10X10X1_2***	**1**	D1 at 100.00%	0.55	OK	
		7	D1 at 100.00%	**0.71**	**OK**	
	HSS_SQR 10X10X3_8	**3**	D4 at 100.00%	0.06	OK	
		4	D1 at 100.00%	**0.36**	**OK**	
		34	D1 at 0.00%	0.12	OK	

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Specifier's comments:

1 Input data



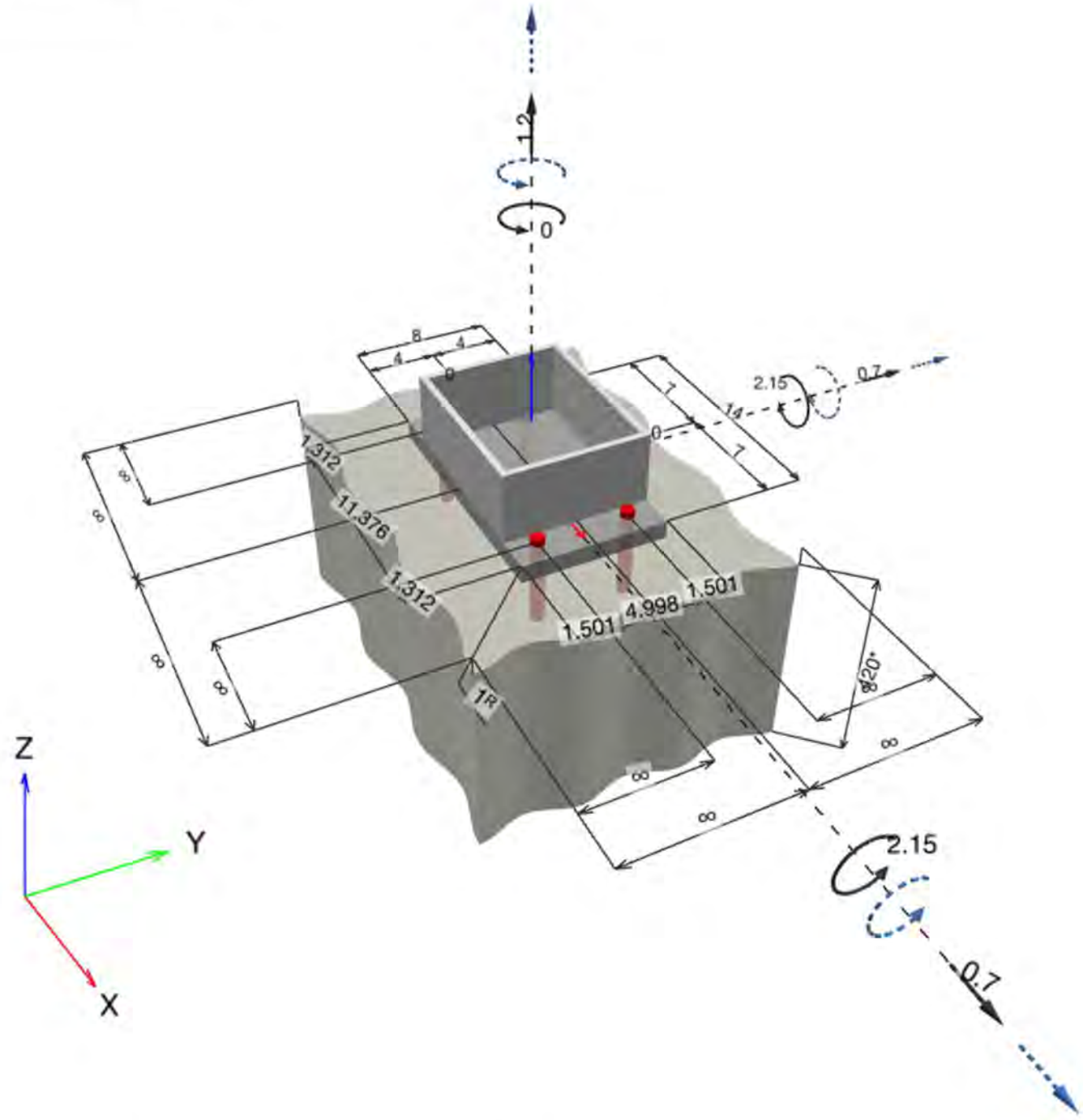
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418085 KH-EZ 3/4"x7"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	3/1/2026 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R]:	l_x x l_y x t = 14.000 in. x 8.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X8X.375; (L x W x T) = 10.000 in. x 8.000 in. x 0.375 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 420.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

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Geometry [in.] & Loading [kip, ft.kip]


12
0
8
4
4
0
7
14
0
7
2.15
0.7
1.312
11.376
1.312
1.501
4.998
1.501
8
8
8

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1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	-	-	1.000	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	-	-	-	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	-	-	-	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	-	0.700	-	-	-	-
E (Earthquake)	-	-	0.700	0.700	-	2.15000	2.15000	-

1.2 Load combination and design results

1.2.1 Load combination

Load case	Load combination
Equation (16-1)	1.4 (D + F)
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.2.2 Design results

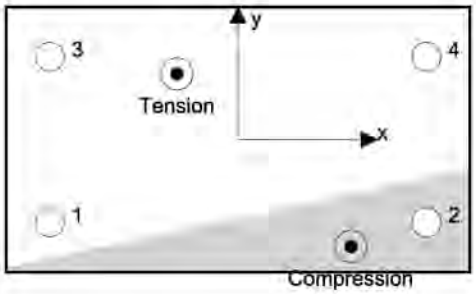
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
Equation (16-1)	1.4 (D + F)	N = 1.400; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	10
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_2L + 1.6 (H) + f_2S	N = 1.200; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	56
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)	N = 0.900; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	7
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)	N = 0.900; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	54

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

Controlling load case: Equation (16-5) 1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.590	0.247	0.175	0.175
2	0.000	0.247	0.175	0.175
3	3.181	0.247	0.175	0.175
4	1.922	0.247	0.175	0.175

Max. concrete compressive strain: 0.14 [‰]
Max. concrete compressive stress: 610 [psi]
Resulting tension force in (x/y)=(-1.847/1.981): 5.693 [kip]
Resulting compression force in (x/y)=(3.402/-3.232): 4.493 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	3.181	20.808	16	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	5.693	10.294	56	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
32.013	0.650	20.808	3.181

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\, N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\, \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.770	0.049	1.148	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.690	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
312.36	127.92	0.991	0.831	1.000	1.000	7.870

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\, N_{cbg}$ [kip]	N_{ua} [kip]
15.837	0.650	10.294	5.693

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.247	9.996	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.990	31.775	4	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [kip]
16.660

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.660	0.600	9.996	0.247

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.690	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
368.89	127.92	1.000	1.000	1.000	1.000	7.870

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
45.393	0.700	31.775	0.990

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.553	0.031	5/3	38	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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7 Installation data

Profile: Rectangular HSS (AISC), HSS10X8X.375; (L x W x T) = 10.000 in. x 8.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)

Item number: 418085 KH-EZ 3/4"x7"

Maximum installation torque: 0.09501 ft.kip

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.375 in.

Minimum thickness of the base material: 7.000 in.

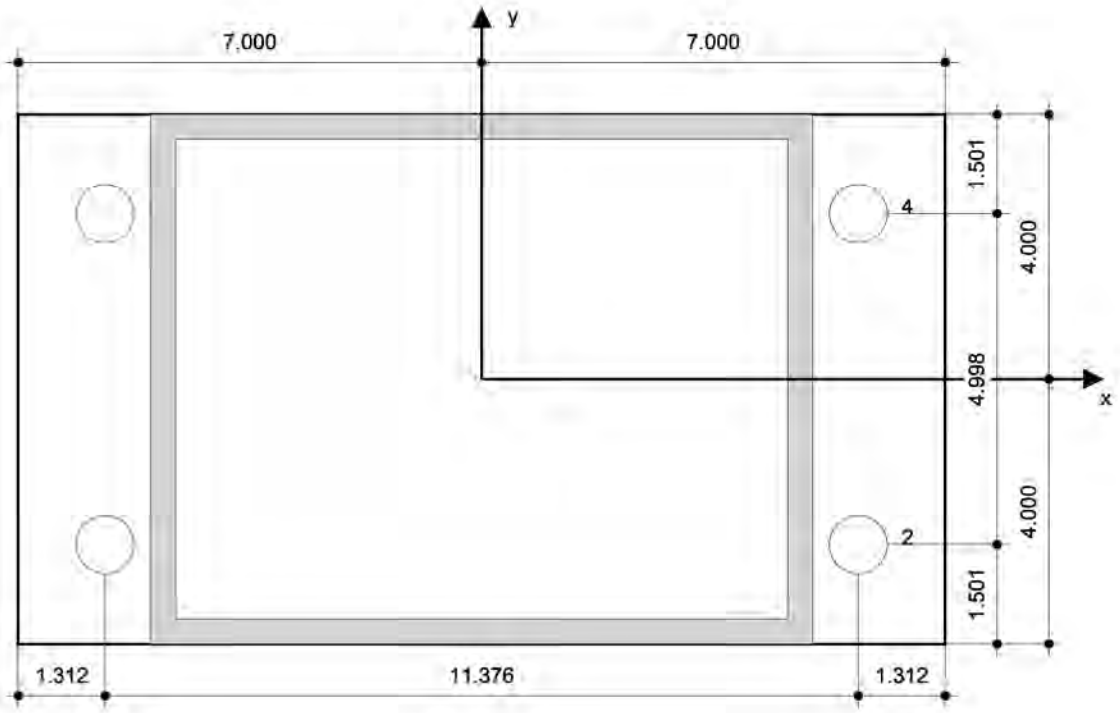
Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 5 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hilti SIW 9-A22 Impact Wrench

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.688	-2.499	-	-	-	-
2	5.688	-2.499	-	-	-	-
3	-5.688	2.499	-	-	-	-
4	5.688	2.499	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Specifier's comments:

1 Input data



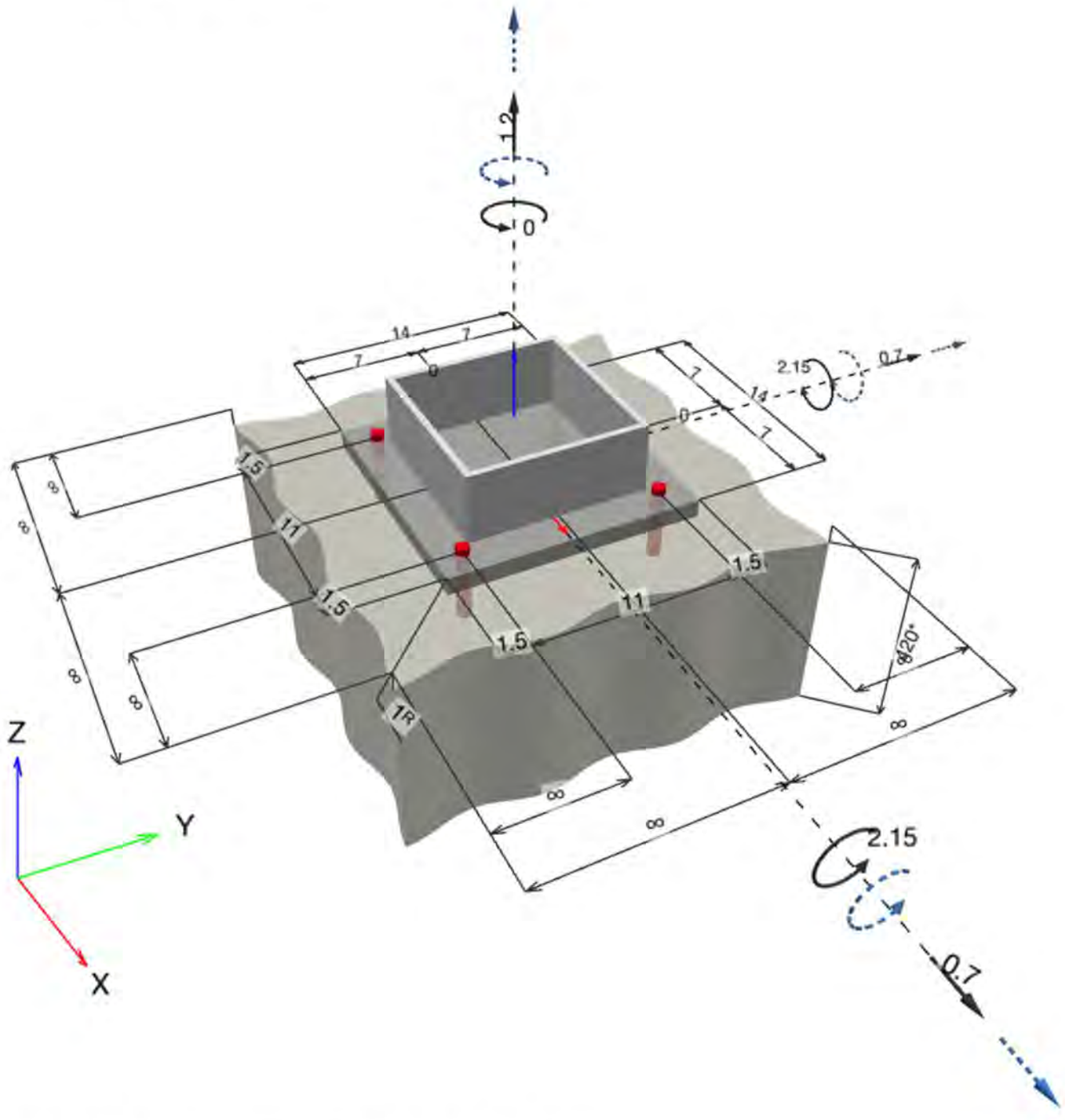
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (4)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 4 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 2.920 in., h_{nom} = 4.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	3/1/2026 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R]:	l_x x l_y x t = 14.000 in. x 14.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS10X10X.375; (L x W x T) = 10.000 in. x 10.000 in. x 0.375 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 420.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	-	-	1.000	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	-	-	-	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	-	-	-	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	-	0.700	-	1.92000	1.92000	-
E (Earthquake)	-	-	0.700	0.700	-	2.15000	2.15000	-

1.2 Load combination and design results

1.2.1 Load combination

Load case	Load combination
Equation (16-1)	1.4 (D + F)
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.2.2 Design results

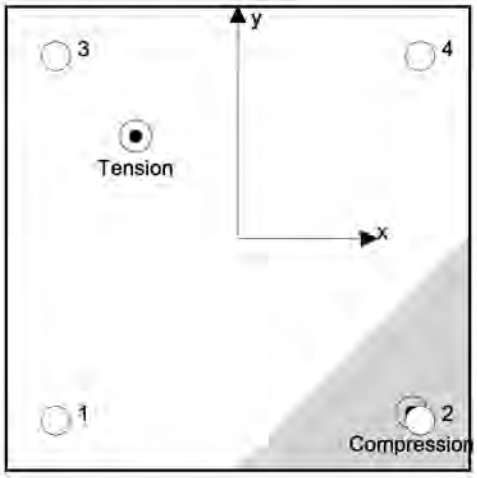
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
Equation (16-1)	1.4 (D + F)	N = 1.400; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	11
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_2L + 1.6 (H) + f_2S	N = 1.200; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	62
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)	N = 0.900; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	54
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)	N = 0.900; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	59

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

Controlling load case: Equation (16-5) 1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S



Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.841	0.247	0.175	0.175
2	0.000	0.247	0.175	0.175
3	2.165	0.247	0.175	0.175
4	0.841	0.247	0.175	0.175

Max. concrete compressive strain: 0.07 [‰]
Max. concrete compressive stress: 323 [psi]
Resulting tension force in (x/y)=(-3.095/3.095): 3.847 [kip]
Resulting compression force in (x/y)=(5.247/-5.247): 2.647 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	2.165	20.808	11	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	3.847	6.305	62	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
32.013	0.650	20.808	2.165

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.920	1.262	1.262	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.410	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
230.21	76.74	0.776	0.776	1.000	1.000	5.365

Results

N_{cbg} [kip]	$\phi_{concrete}$	ϕN_{cbg} [kip]	N_{ua} [kip]
9.700	0.650	6.305	3.847

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.247	9.996	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.990	30.043	4	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [kip]
16.660

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.660	0.600	9.996	0.247

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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	2.920	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	4.410	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
306.95	76.74	1.000	1.000	1.000	1.000	5.365

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
42.918	0.700	30.043	0.990

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.610	0.033	5/3	45	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Electrical Room - HSS10x10	Date:	8/17/2026
Fastening point:			

7 Installation data

Profile: Square HSS (AISC), HSS10X10X.375; (L x W x T) = 10.000 in. x 10.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (4)

Item number: 418084 KH-EZ 3/4"x5 1/2"

Maximum installation torque: 0.09501 ft.kip

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 4.375 in.

Minimum thickness of the base material: 6.000 in.

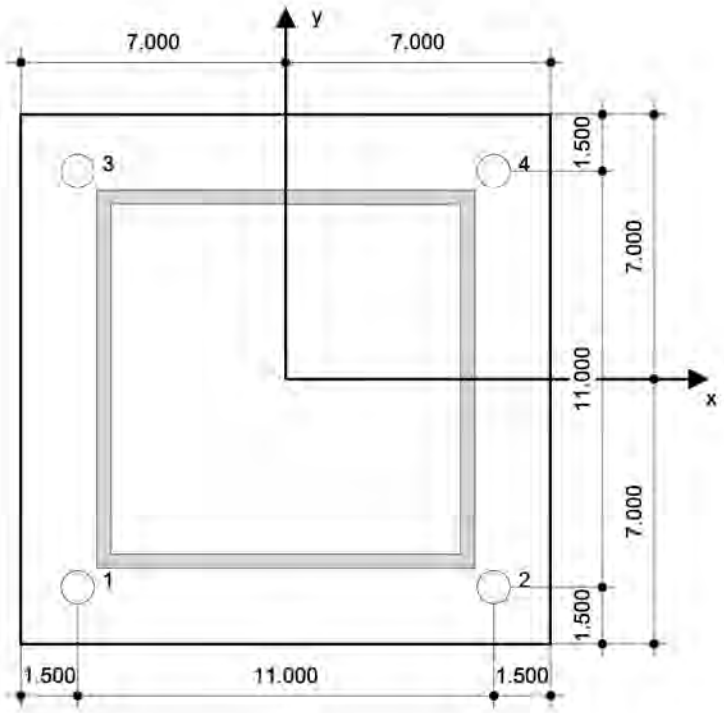
Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 4 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hilti SIW 9-A22 Impact Wrench

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Electrical Room - HSS10x10	Date:	8/17/2026
Fastening point:			



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.500	-5.500	-	-	-	-
2	5.500	-5.500	-	-	-	-
3	-5.500	5.500	-	-	-	-
4	5.500	5.500	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

LIC# : KW-06018138, Build:20.26.05.05 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Mechanical - Worst Case

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : ACI 318-19
Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	0.950 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

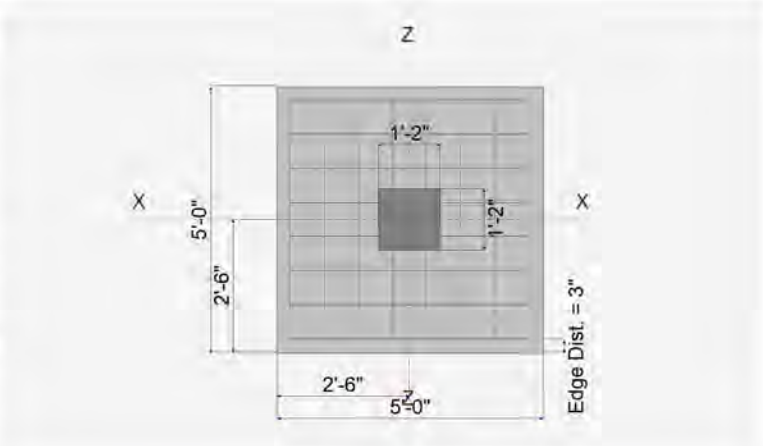
Width parallel to X-X Axis	=	5.0 ft
Length parallel to Z-Z Axis	=	5.0 ft
Footing Thickness	=	8.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	14.0 in
pz : parallel to Z-Z Axis	=	14.0 in
Height	=	1.0 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=		8.0
Reinforcing Bar Size	=	#	5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=		8.0
Reinforcing Bar Size	=	#	5

Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in



Top Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=	5.0
Reinforcing Bar Size	=	5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	5.0
Reinforcing Bar Size	=	5

Rebar Centerline to Edge of Concrete...
at Top of footing = 1.50 in

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

LIC# : KW-06018138, Build:20.26.05.05 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Mechanical - Worst Case

Applied Loads

		D	Lr	L	S	W	E	H
P : Vertical Load	=	0.000	0.000	47.900	0.000	0.000	0.000	0.000 k
OB : Overburden	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 ksf
M-xx	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
M-zz	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
V-x	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k
V-z	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8052	Soil Bearing	2.013 ksf	2.50 ksf	+D+L about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	999.0	Sliding - X-X	0.0 k	1.239 k	D Only
PASS	999.0	Sliding - Z-Z	0.0 k	1.239 k	D Only
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.5442	Z Flexure (+X) Bot Tens	5.630 k-ft/ft	10.346 k-ft/ft	+1.20D+1.60L
PASS	0.5442	Z Flexure (-X) Bot Tens	5.630 k-ft/ft	10.346 k-ft/ft	+1.20D+1.60L
PASS	0.348	Min Steel X Flexure Bottom	0.173 in2/ft	0.496 in2/ft	n/a
PASS	0.5442	X Flexure (+Z) Bot Tens	5.630 k-ft/ft	10.346 k-ft/ft	+1.20D+1.60L
PASS	0.5442	X Flexure (-Z) Bot Tens	5.630 k-ft/ft	10.346 k-ft/ft	+1.20D+1.60L
PASS	0.348	Min Steel Z Flexure Bottom	0.173 in2/ft	0.496 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.9988	1-way Shear (+X)	76.640 psi	76.729 psi	+1.20D+1.60L
PASS	0.9988	1-way Shear (-X)	76.640 psi	76.729 psi	+1.20D+1.60L
PASS	0.9988	1-way Shear (+Z)	76.640 psi	76.729 psi	+1.20D+1.60L
PASS	0.9988	1-way Shear (-Z)	76.640 psi	76.729 psi	+1.20D+1.60L
PASS	0.9541	2-way Punching	181.032 psi	189.737 psi	+1.20D+1.60L

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.09667	0.09667	n/a	n/a	0.039
X-X, +D+L	2.50	n/a	0.0	2.013	2.013	n/a	n/a	0.805
X-X, +D+0.750L	2.50	n/a	0.0	1.534	1.534	n/a	n/a	0.614
X-X, +0.60D	2.50	n/a	0.0	0.0580	0.0580	n/a	n/a	0.023
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.09667	0.09667	0.039
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	2.013	2.013	0.805
Z-Z, +D+0.750L	2.50	0.0	n/a	n/a	n/a	1.534	1.534	0.614
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.0580	0.0580	0.023

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft) Overturning	Resisting	Ratio	Status	Moment Sum About Positive Edge (k-ft) Overturning	Resisting	Ratio	Status
Footing Has NO Overturning								

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

LIC# : KW-06018138, Build:20.26.05.05 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Mechanical - Worst Case

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.0	+Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +1.40D	0.0	-Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +1.20D+1.60L	5.630	+Z	Bottom	0.2602	Applied Mu	0.4960	10.346	OK
X-X, +1.20D+1.60L	5.630	-Z	Bottom	0.2602	Applied Mu	0.4960	10.346	OK
X-X, +1.20D+0.50L	1.760	+Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +1.20D+0.50L	1.760	-Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +1.20D	0.0	+Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +1.20D	0.0	-Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +0.90D	0.0	+Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
X-X, +0.90D	0.0	-Z	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.40D	0.0	-X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.40D	0.0	+X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.20D+1.60L	5.630	-X	Bottom	0.2602	Applied Mu	0.4960	10.346	OK
Z-Z, +1.20D+1.60L	5.630	+X	Bottom	0.2602	Applied Mu	0.4960	10.346	OK
Z-Z, +1.20D+0.50L	1.760	-X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.20D+0.50L	1.760	+X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.20D	0.0	-X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +1.20D	0.0	+X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +0.90D	0.0	-X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK
Z-Z, +0.90D	0.0	+X	Bottom	0.1728	ACI 7.6.1.1	0.4960	10.346	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK
+1.20D+1.60L	76.64 psi	76.64 psi	76.64 psi	76.73 psi	1.00	OK
+1.20D+0.50L	23.95 psi	23.95 psi	23.95 psi	76.73 psi	0.31	OK
+1.20D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK

One Way Shear Z

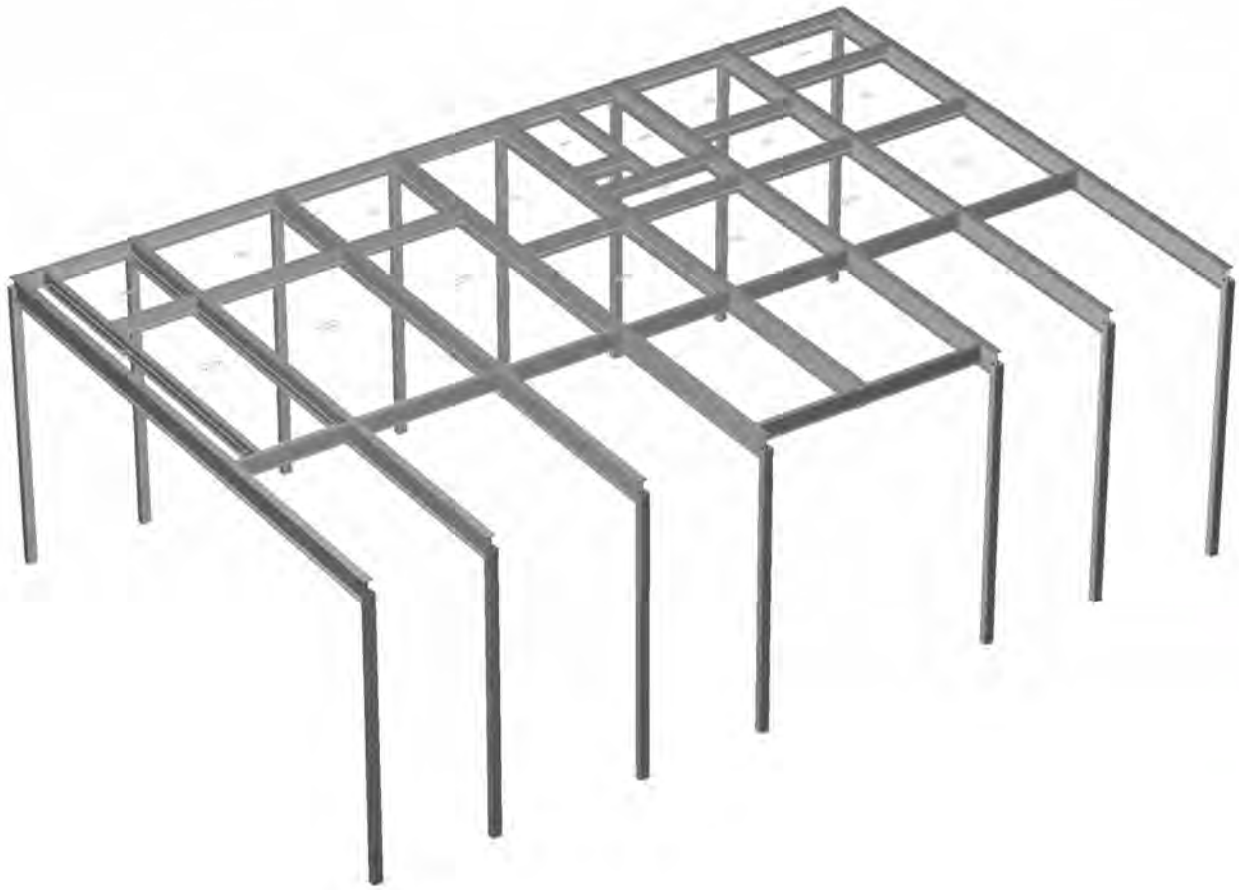
Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK
+1.20D+1.60L	76.64 psi	76.64 psi	76.64 psi	76.73 psi	1.00	OK
+1.20D+0.50L	23.95 psi	23.95 psi	23.95 psi	76.73 psi	0.31	OK
+1.20D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	76.73 psi	0.00	OK

Two-Way "Punching" Shear

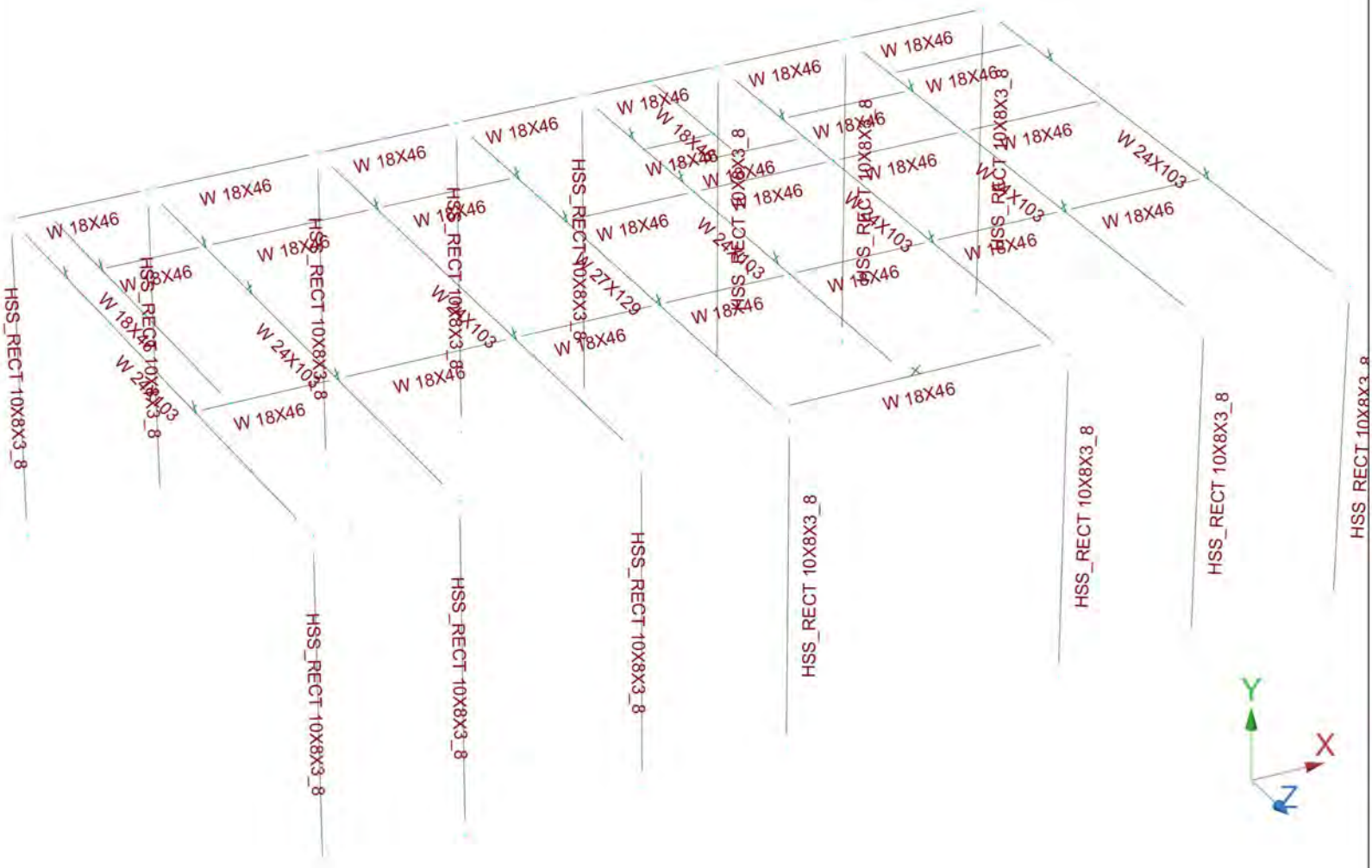
All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	189.74 psi	0.00	OK
+1.20D+1.60L	181.03 psi	189.74 psi	0.95	OK
+1.20D+0.50L	56.57 psi	189.74 psi	0.30	OK
+1.20D	0.00 psi	189.74 psi	0.00	OK
+0.90D	0.00 psi	189.74 psi	0.00	OK

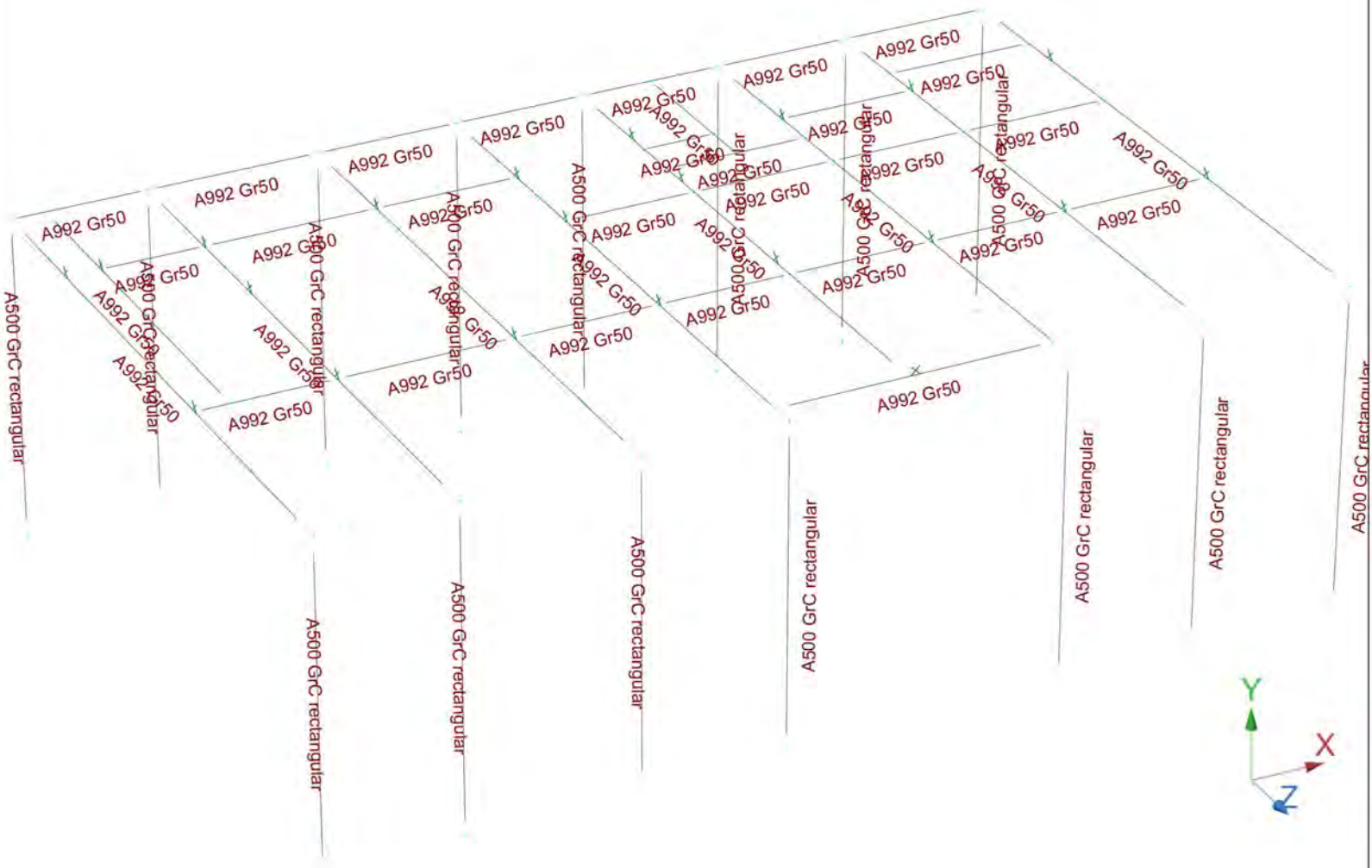
Electrical Room Equipment Support Framing
3D Rendering



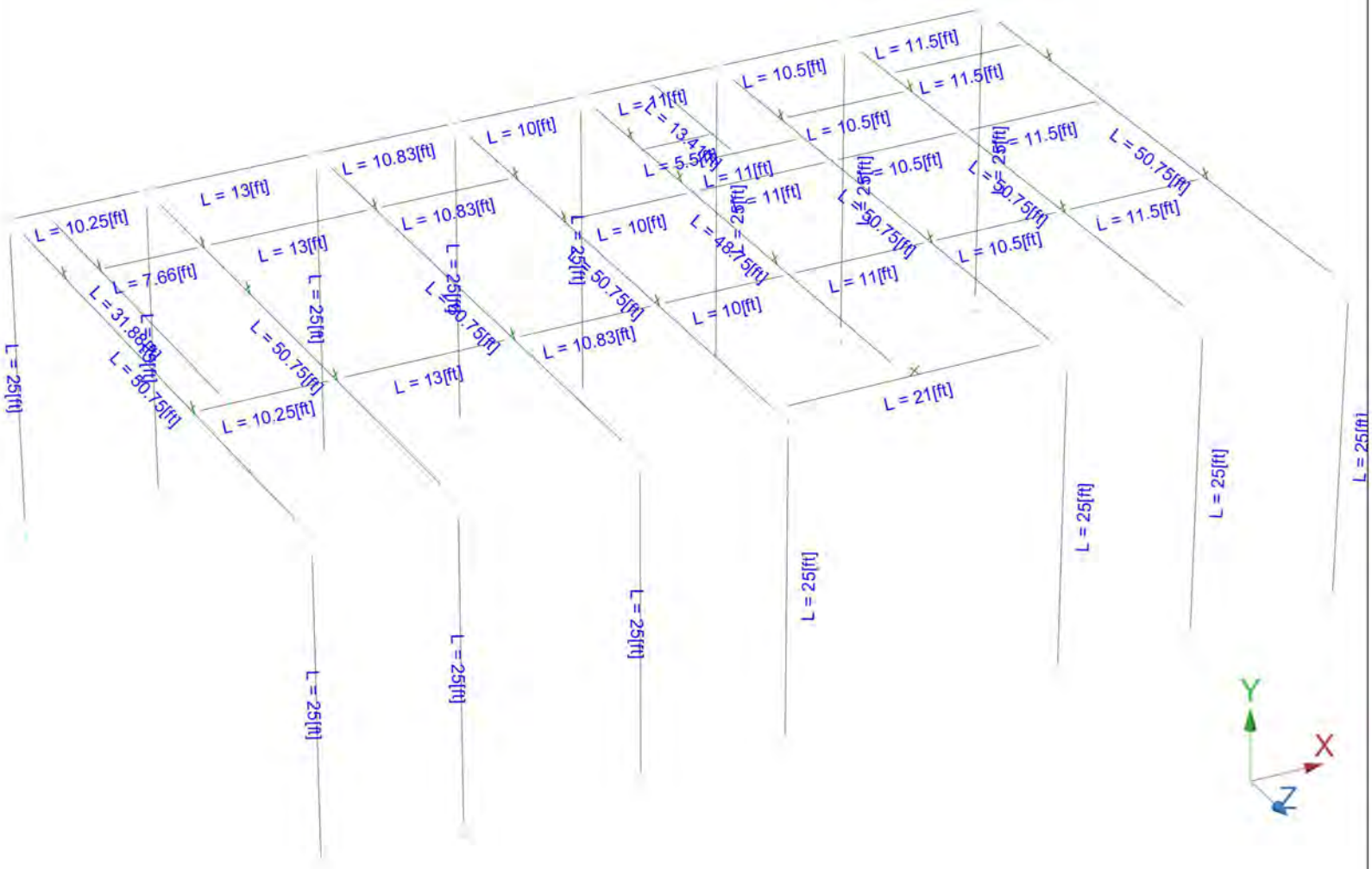
Electrical Room Equipment Support Framing Member Sizes



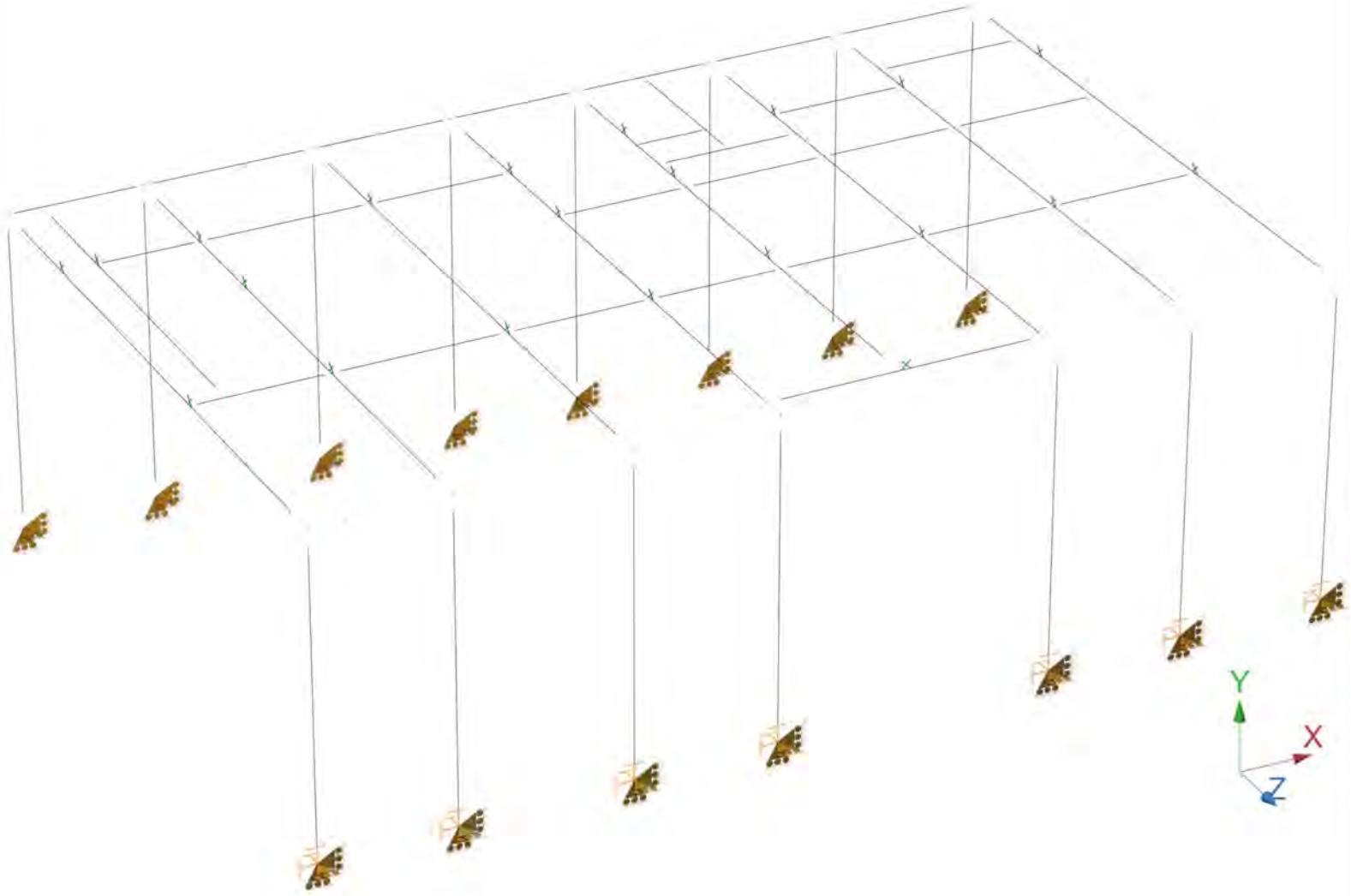
Electrical Room Equipment Support Framing Member Material



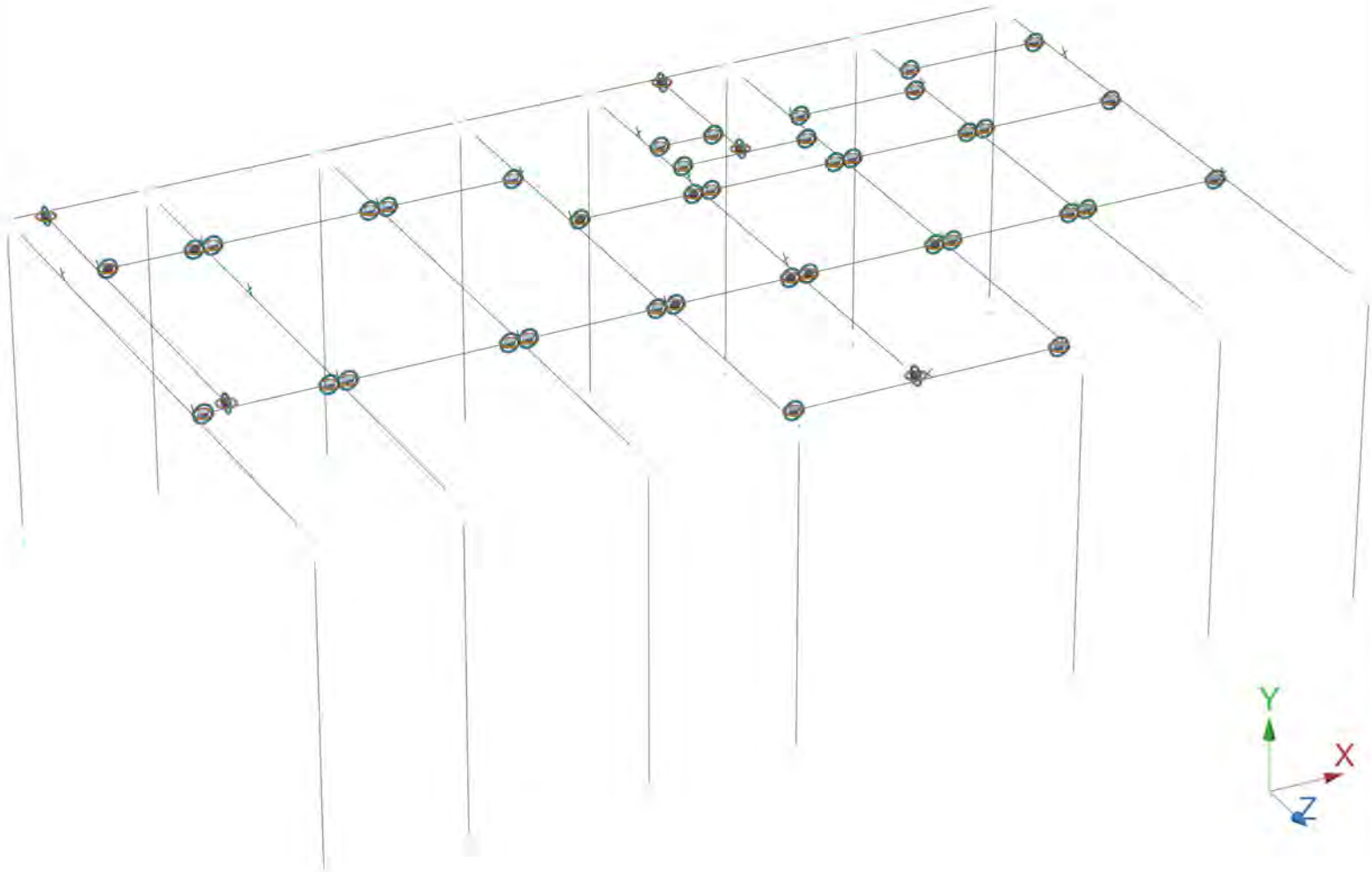
Electrical Room Equipment Support Framing Member Lengths



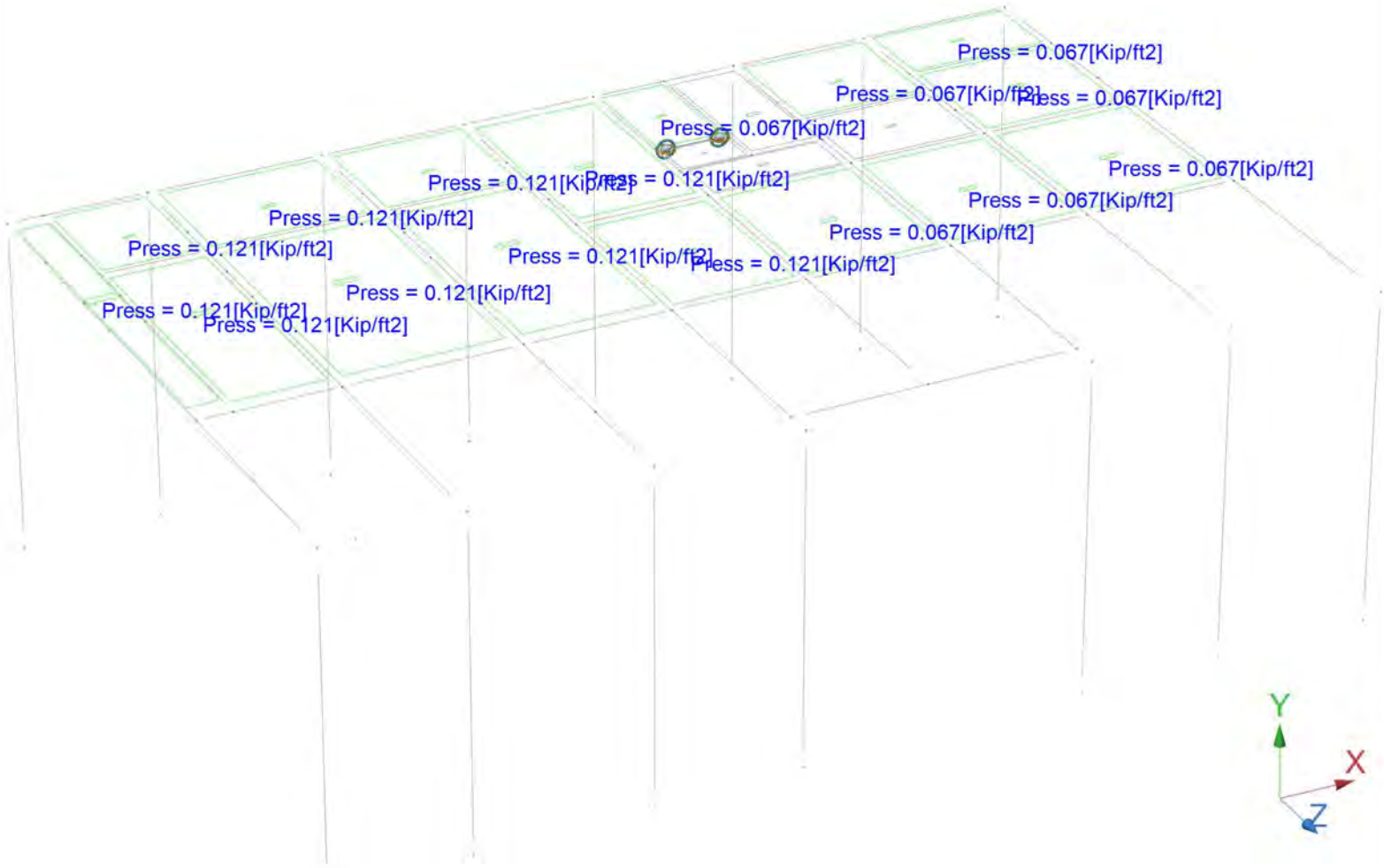
Electrical Room Equipment Support Framing
Column Supports



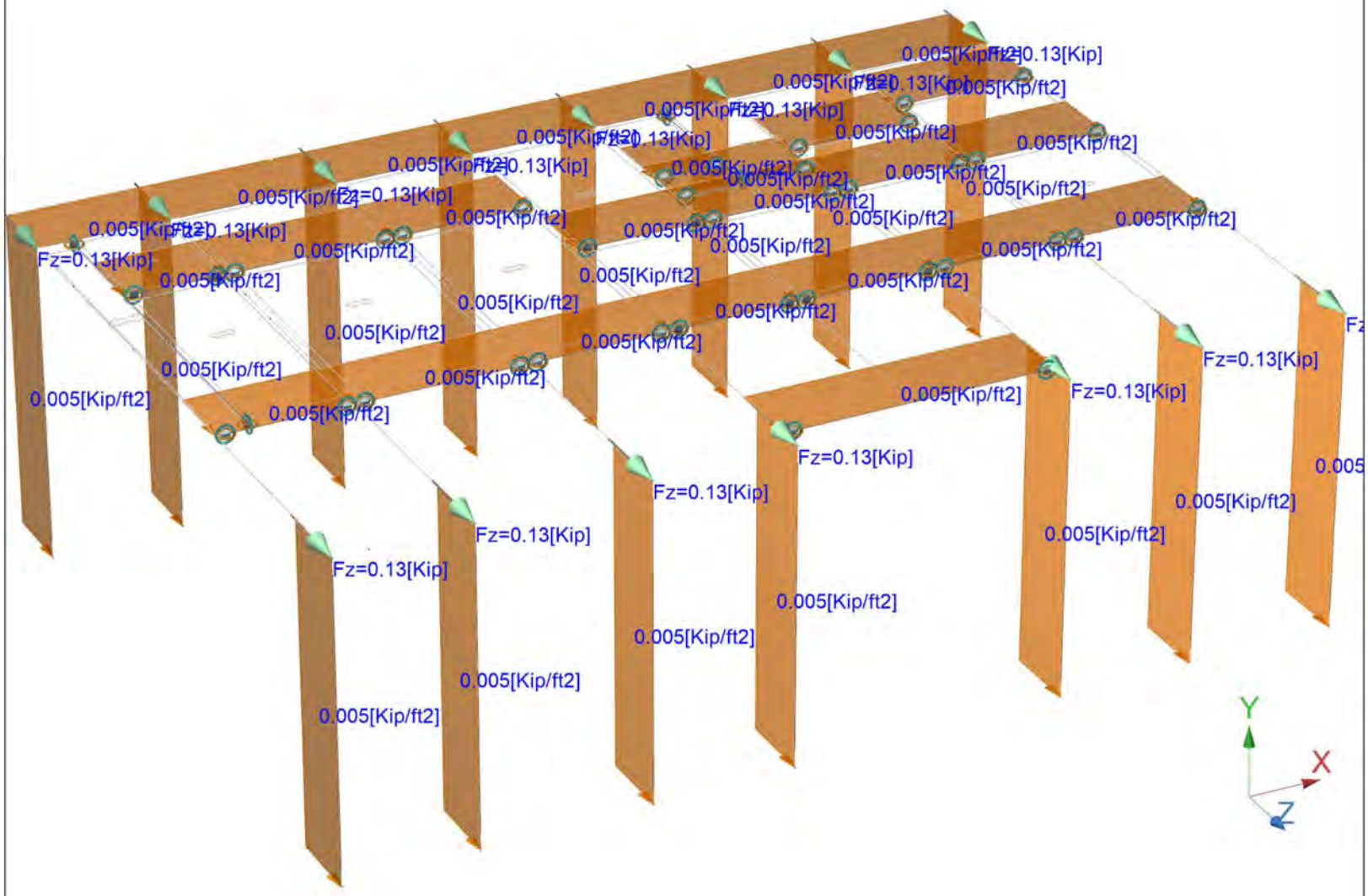
Electrical Room Equipment Support Framing
Beam Hingers



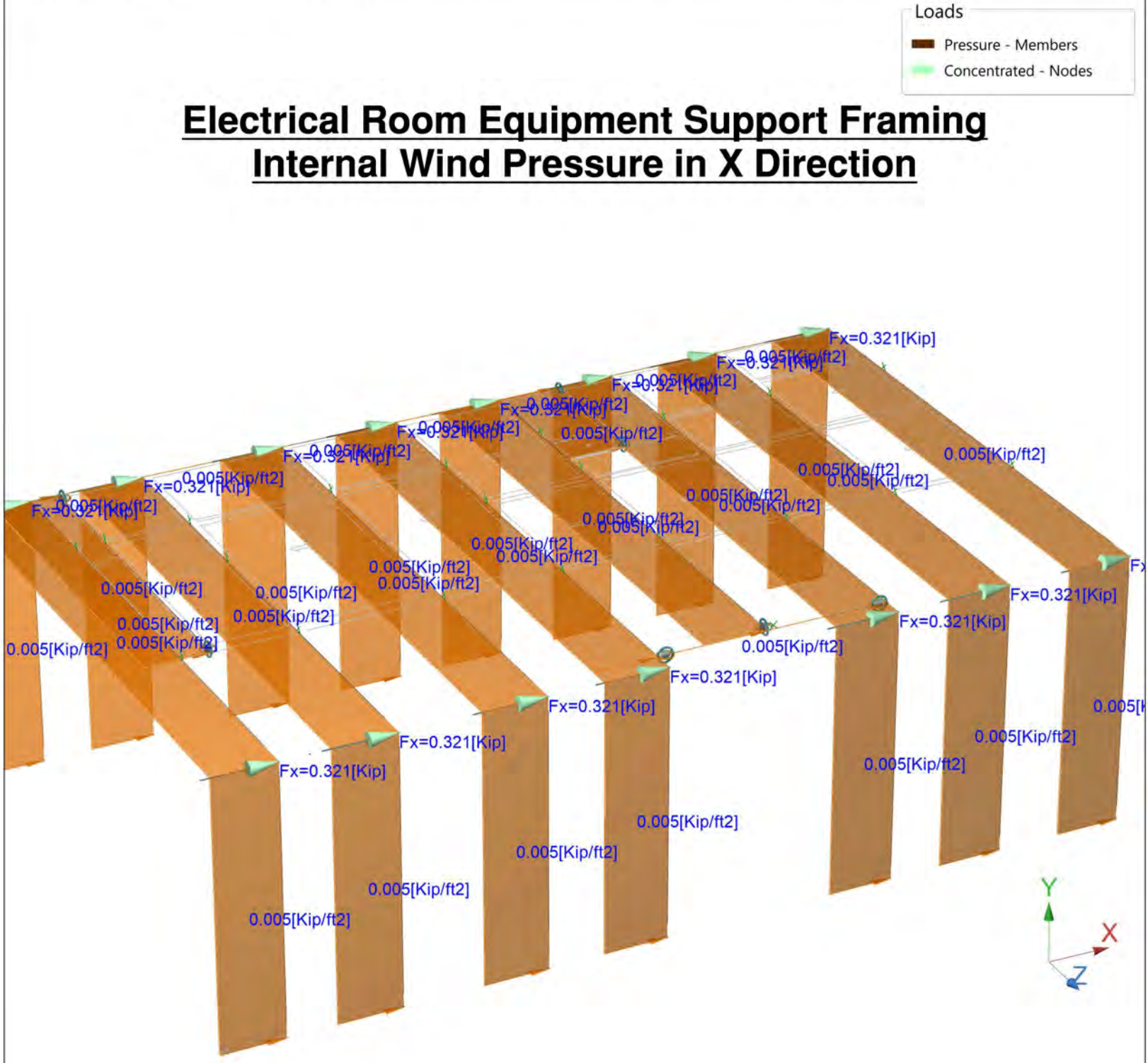
Electrical Room Equipment Support Framing Dead Loads



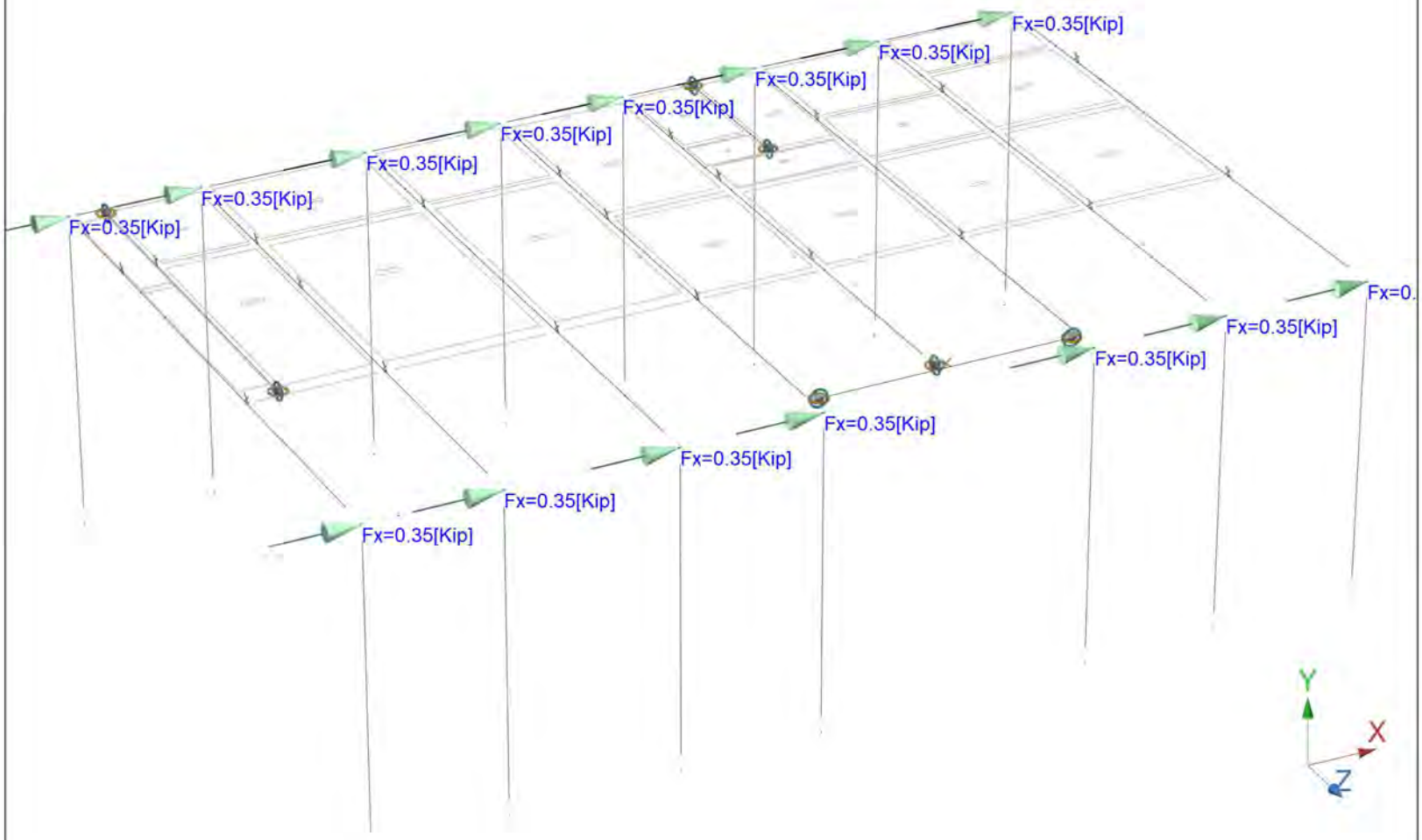
Electrical Room Equipment Support Framing
Internal Wind Pressure in Z Direction



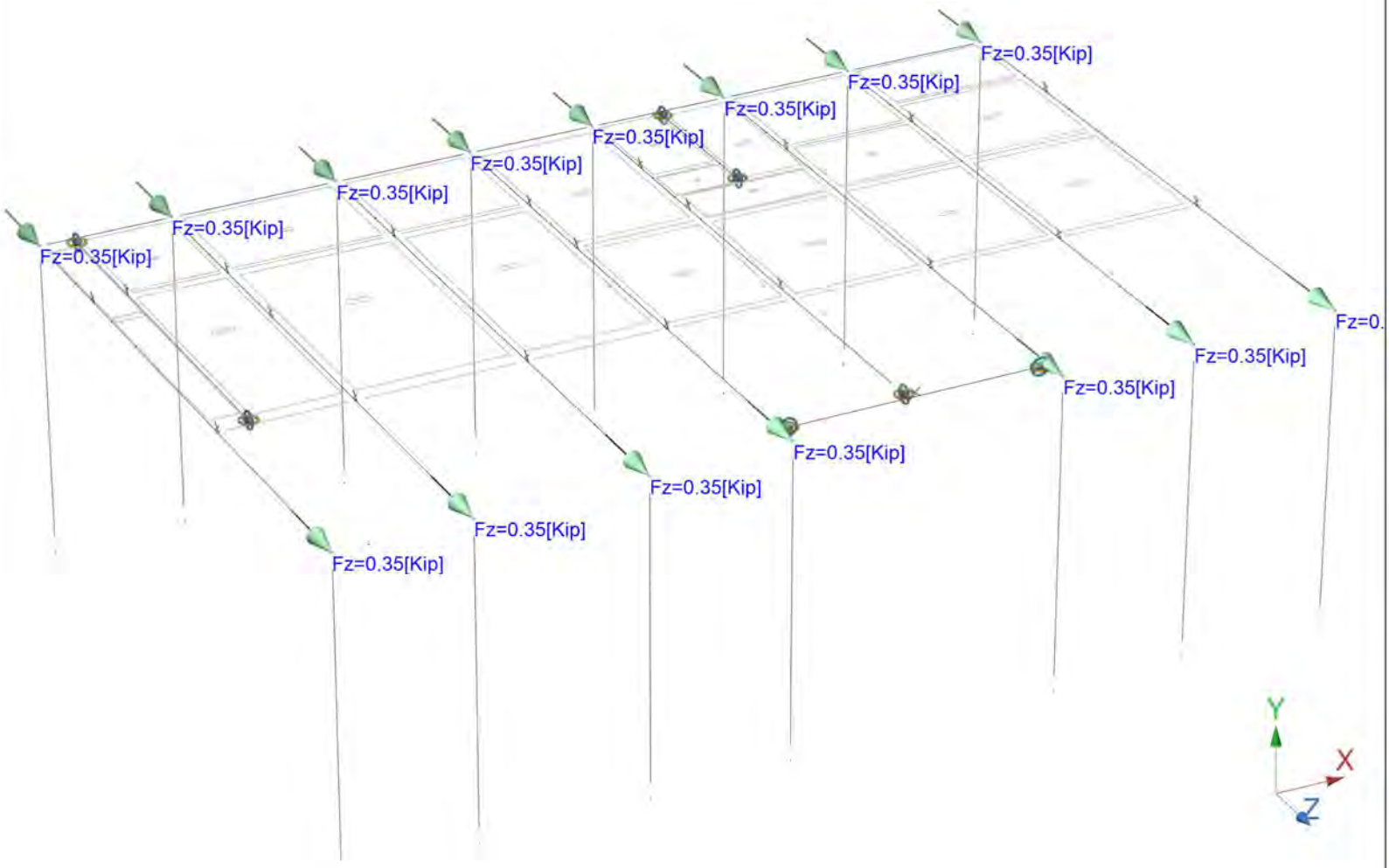
Electrical Room Equipment Support Framing
Internal Wind Pressure in X Direction

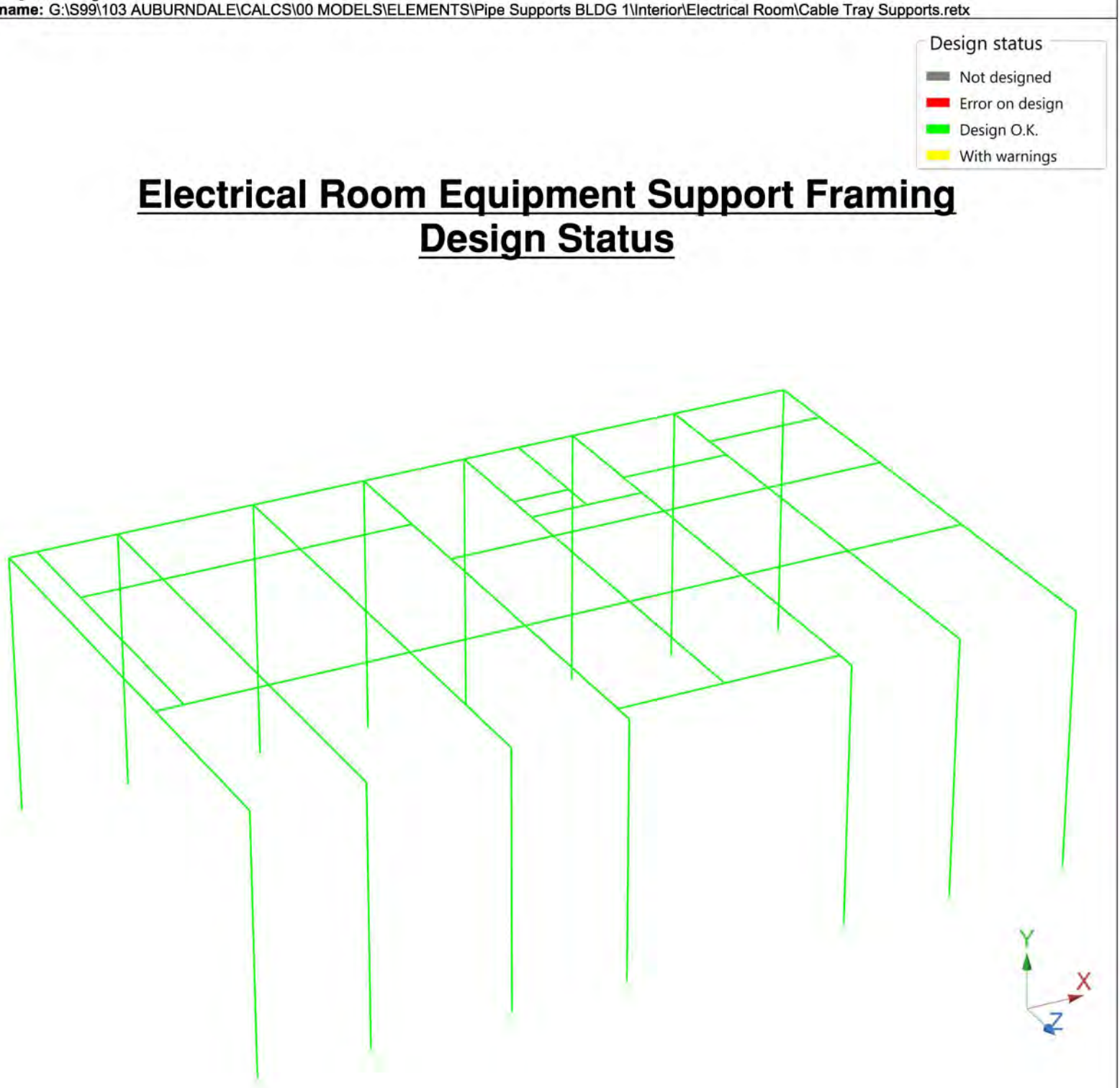


Electrical Room Equipment Support Framing
Lateral Load in X Direction



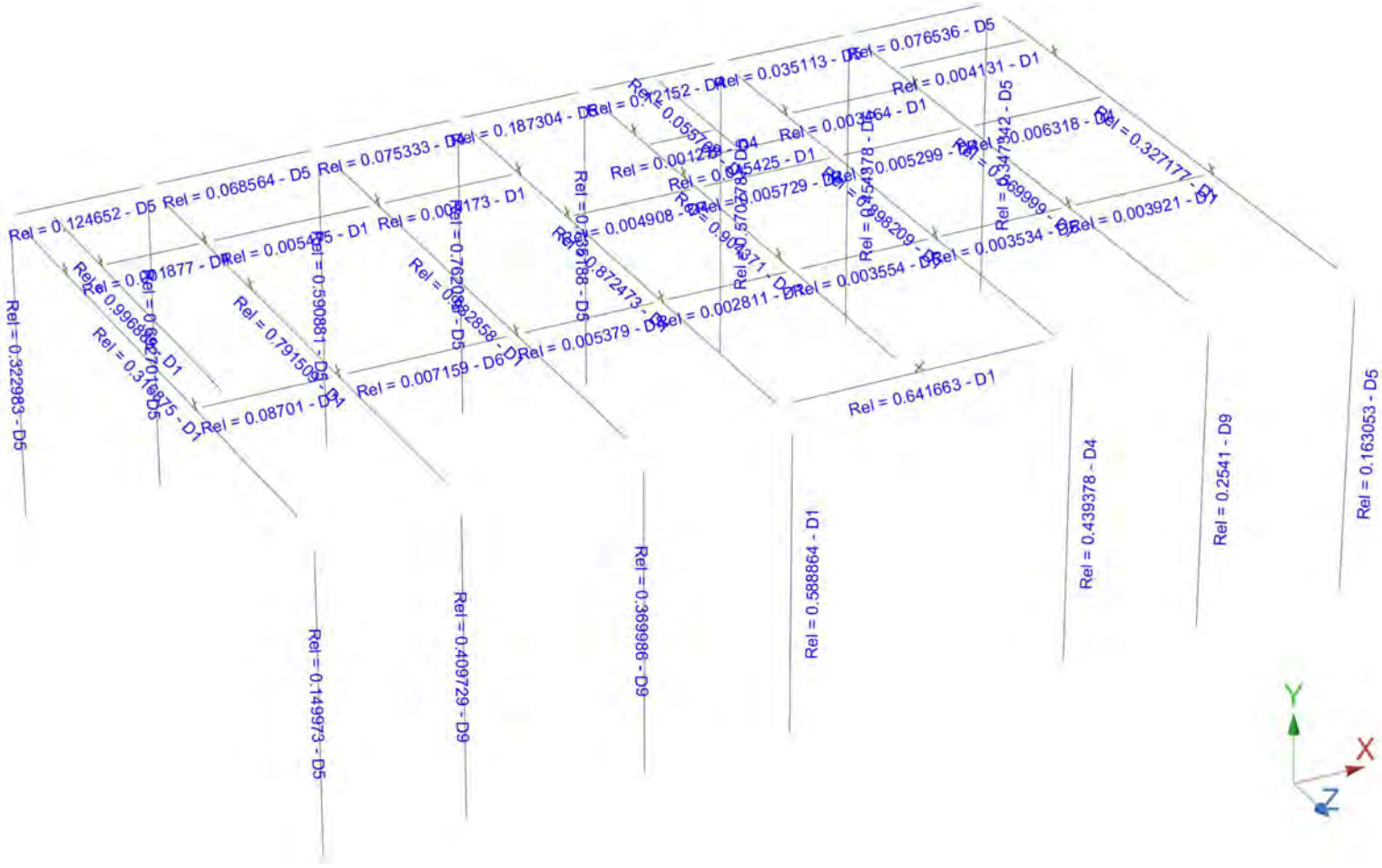
Electrical Room Equipment Support Framing
Lateral Load in Z Direction





Electrical Room Equipment Support Framing
Design Status

Electrical Room Equipment Support Framing
Design Status Ratios



Current Date: 8/17/2026 11:35 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pipe Supports BLDG 1\Interior\Electrical Room\Cable Tray Supports.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5WLz
D3=1.2DL+0.5WLx
D4=1.2DL+WLz
D5=1.2DL+WLx
D6=0.9DL+WLz
D7=0.9DL+WLx
D8=1.2DL+EQx
D9=1.2DL+EQz
D10=0.9DL+EQx
D11=0.9DL+EQz

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam	*W 18X46*	23	D1 at 47.62%	0.64	OK	
		25	D1 at 25.27%	0.09	OK	
		26	D6 at 50.00%	0.01	OK	
		27	D4 at 50.00%	0.01	OK	
		28	D5 at 50.00%	0.00	OK	
		29	D1 at 50.00%	0.00	OK	
		30	D1 at 50.25%	**1.00**	**OK**	
		31	D5 at 25.27%	0.12	OK	
		32	D4 at 50.00%	0.01	OK	
		33	D4 at 50.00%	0.01	OK	
		34	D1 at 50.00%	0.00	OK	
		35	D1 at 50.00%	0.00	OK	
		36	D5 at 50.00%	0.00	OK	
		37	D1 at 50.00%	0.00	OK	
		38	D4 at 50.00%	0.01	OK	
		39	D4 at 50.00%	0.00	OK	
		40	D1 at 50.00%	0.05	OK	
		41	D5 at 100.00%	0.07	OK	
		42	D4 at 0.00%	0.08	OK	
		43	D5 at 0.00%	0.19	OK	
		44	D4 at 0.00%	0.12	OK	
		45	D5 at 100.00%	0.04	OK	
		46	D5 at 100.00%	0.08	OK	
		52	D1 at 49.24%	0.06	OK	
		53	D1 at 50.00%	0.00	OK	
		54	D1 at 50.00%	0.01	OK	
		55	D4 at 75.00%	0.00	OK	
		56	D4 at 50.00%	0.00	OK	
girder	*W 24X103*	16	D1 at 37.18%	0.32	OK	
		17	D1 at 54.76%	0.79	OK	
		18	D1 at 56.51%	**0.93**	**OK**	
		20	D1 at 58.27%	0.90	OK	
		21	D1 at 61.79%	0.57	OK	
		22	D1 at 60.03%	0.33	OK	
		24	D1 at 54.73%	0.90	OK	

	W 27X129	**19**	D1 at 56.51%	**0.87**	**OK**
post	***HSS_RECT 10X8X3_8***	1	D5 at 0.00%	0.15	OK
		2	D9 at 100.00%	0.41	OK
		3	D9 at 100.00%	0.37	OK
		4	D1 at 100.00%	0.59	OK
		5	D4 at 100.00%	0.44	OK
		6	D9 at 100.00%	0.25	OK
		7	D5 at 0.00%	0.16	OK
		8	D5 at 100.00%	0.35	OK
		9	D5 at 100.00%	0.45	OK
		10	D5 at 100.00%	0.57	OK
		11	D5 at 100.00%	0.74	OK
		12	D5 at 100.00%	**0.76**	**OK**
		13	D5 at 100.00%	0.59	OK
		14	D5 at 100.00%	0.69	OK
		15	D5 at 100.00%	0.32	OK

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Specifier's comments:

1 Input data



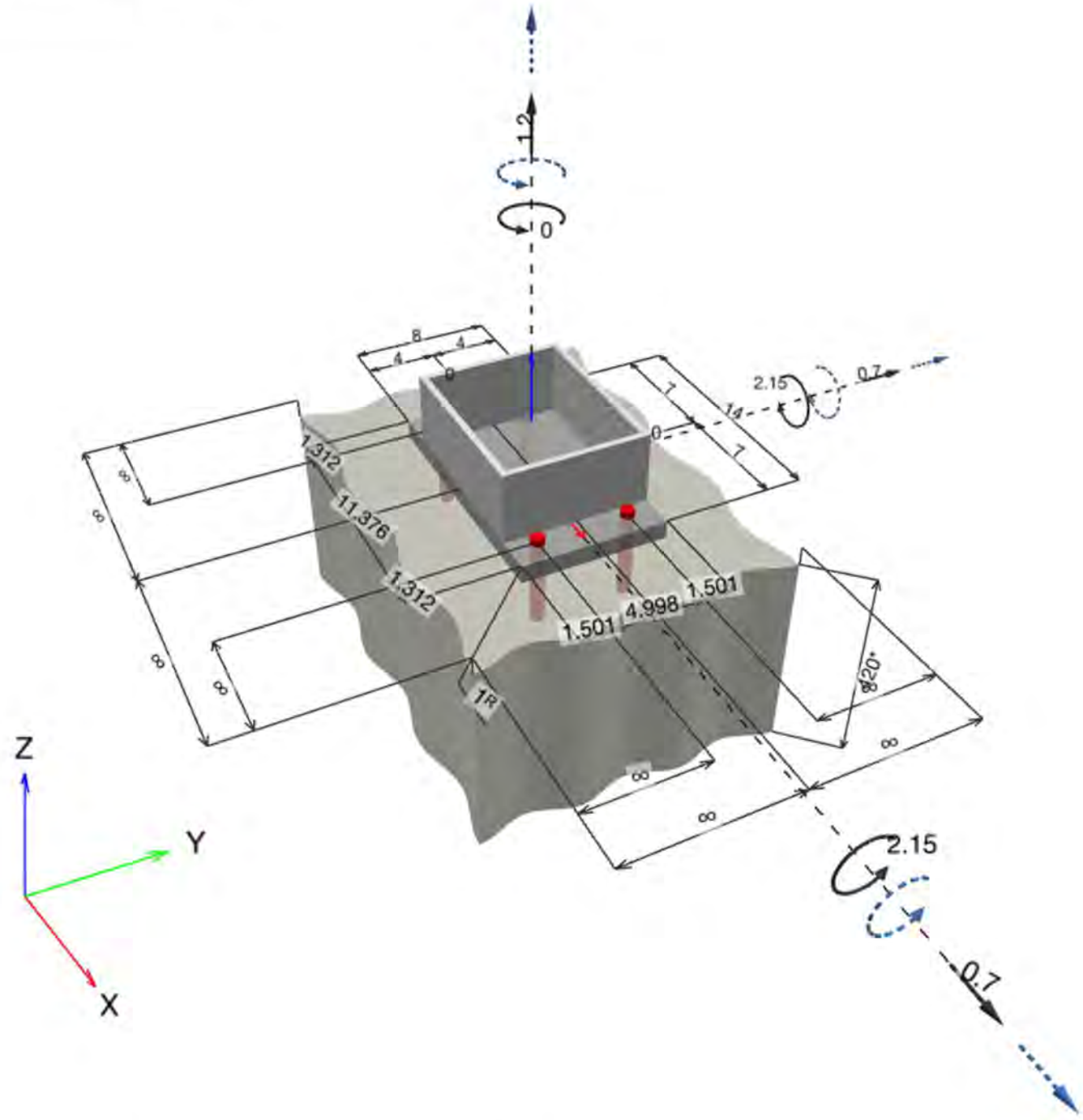
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418085 KH-EZ 3/4"x7"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	3/1/2026 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R]:	l_x x l_y x t = 14.000 in. x 8.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X8X.375; (L x W x T) = 10.000 in. x 8.000 in. x 0.375 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 420.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	-	-	1.000	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	-	-	-	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	-	-	-	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	-	0.700	-	-	-	-
E (Earthquake)	-	-	0.700	0.700	-	2.15000	2.15000	-

1.2 Load combination and design results

1.2.1 Load combination

Load case	Load combination
Equation (16-1)	1.4 (D + F)
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.2.2 Design results

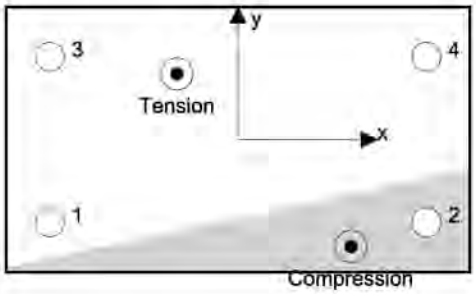
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
Equation (16-1)	1.4 (D + F)	N = 1.400; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	10
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_2L + 1.6 (H) + f_2S	N = 1.200; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	56
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)	N = 0.900; V_x = 0.000; V_y = 0.700; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	7
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)	N = 0.900; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	54

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

Controlling load case: Equation (16-5) 1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.590	0.247	0.175	0.175
2	0.000	0.247	0.175	0.175
3	3.181	0.247	0.175	0.175
4	1.922	0.247	0.175	0.175

Max. concrete compressive strain: 0.14 [‰]
Max. concrete compressive stress: 610 [psi]
Resulting tension force in (x/y)=(-1.847/1.981): 5.693 [kip]
Resulting compression force in (x/y)=(3.402/-3.232): 4.493 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	3.181	20.808	16	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	5.693	10.294	56	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
32.013	0.650	20.808	3.181

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\, N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\, \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.770	0.049	1.148	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.690	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
312.36	127.92	0.991	0.831	1.000	1.000	7.870

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\, N_{cbg}$ [kip]	N_{ua} [kip]
15.837	0.650	10.294	5.693

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.247	9.996	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.990	31.775	4	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [kip]
16.660

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.660	0.600	9.996	0.247

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.690	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
368.89	127.92	1.000	1.000	1.000	1.000	7.870

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
45.393	0.700	31.775	0.990

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.553	0.031	5/3	38	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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7 Installation data

Profile: Rectangular HSS (AISC), HSS10X8X.375; (L x W x T) = 10.000 in. x 8.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)

Item number: 418085 KH-EZ 3/4"x7"

Maximum installation torque: 0.09501 ft.kip

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.375 in.

Minimum thickness of the base material: 7.000 in.

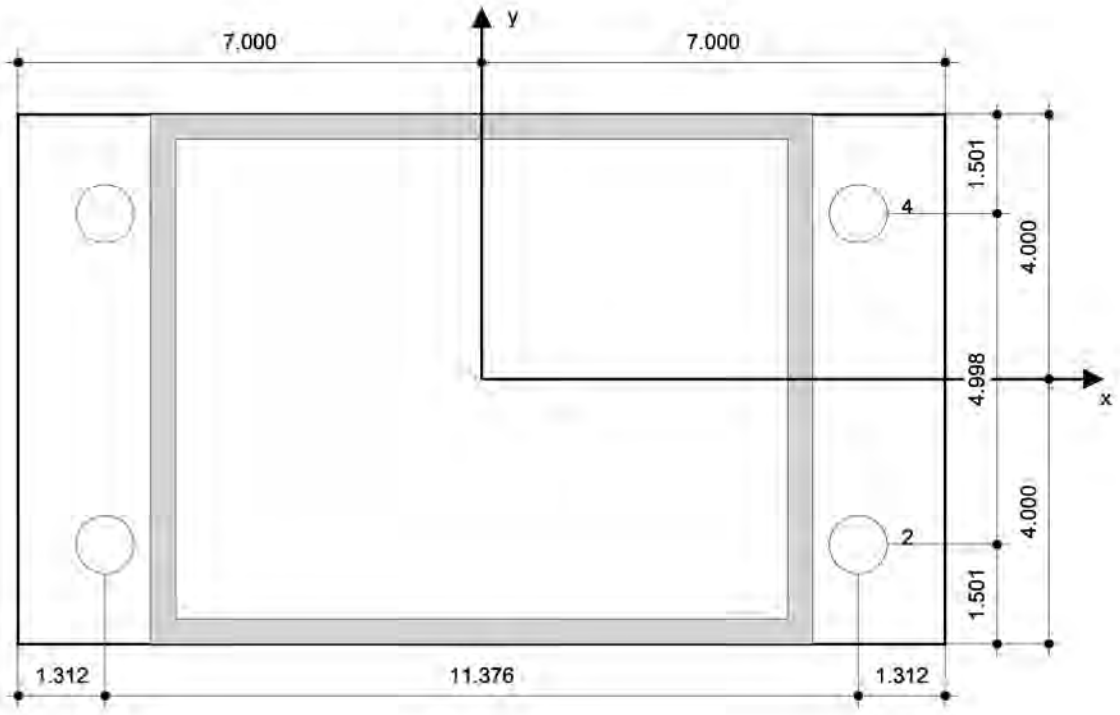
Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 5 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hilti SIW 9-A22 Impact Wrench

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.688	-2.499	-	-	-	-
2	5.688	-2.499	-	-	-	-
3	-5.688	2.499	-	-	-	-
4	5.688	2.499	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Specifier's comments:

1 Input data



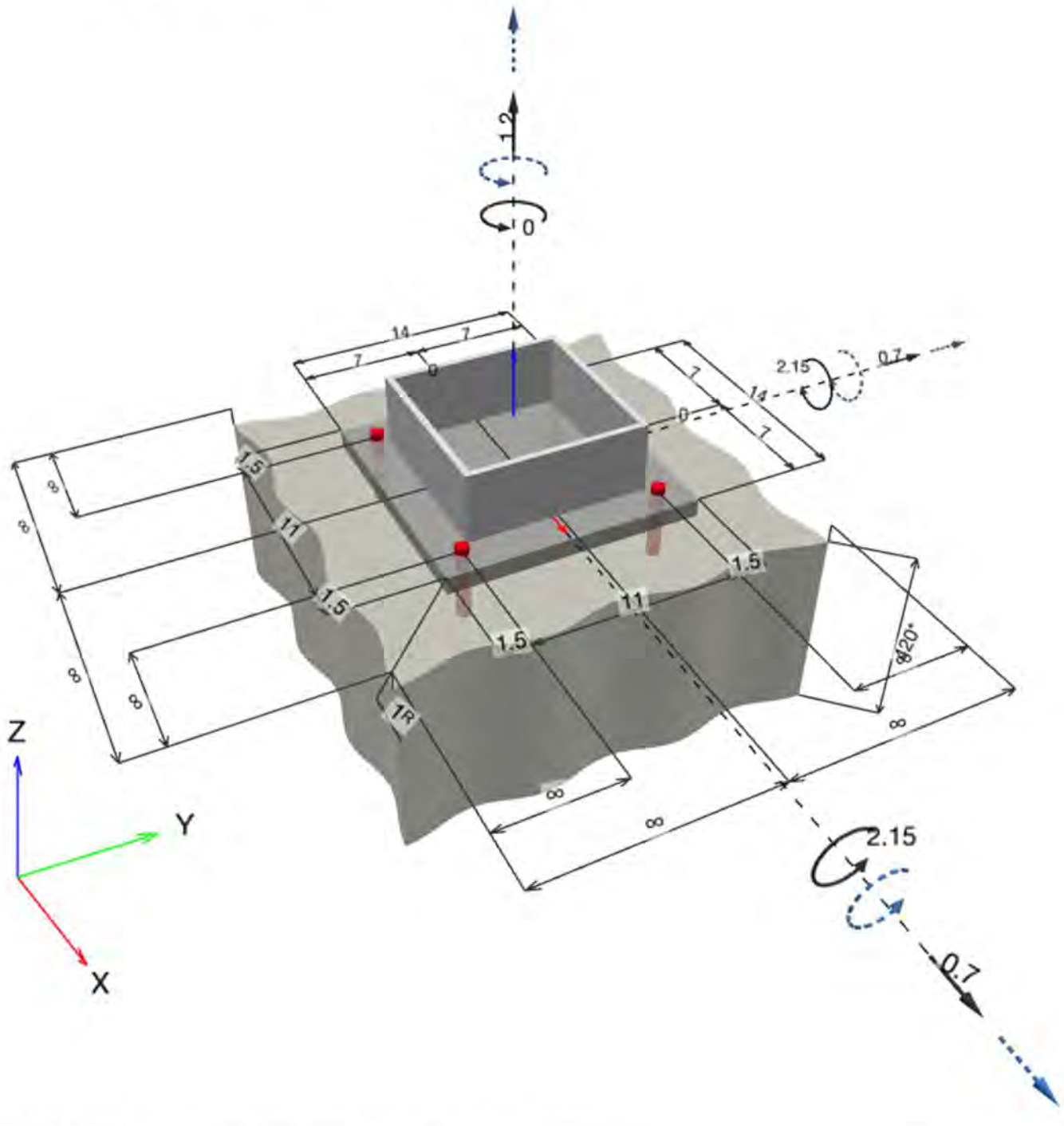
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (4)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 4 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 2.920 in., h_{nom} = 4.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	3/1/2026 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R]:	l_x x l_y x t = 14.000 in. x 14.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS10X10X.375; (L x W x T) = 10.000 in. x 10.000 in. x 0.375 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 420.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	-	-	1.000	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	-	-	-	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	-	-	-	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	-	0.700	-	1.92000	1.92000	-
E (Earthquake)	-	-	0.700	0.700	-	2.15000	2.15000	-

1.2 Load combination and design results

1.2.1 Load combination

Load case	Load combination
Equation (16-1)	1.4 (D + F)
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.2.2 Design results

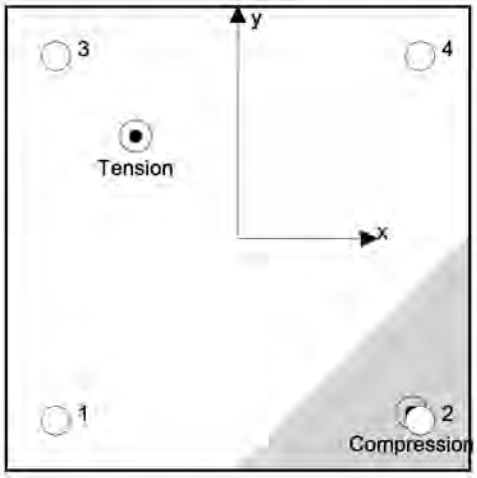
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
Equation (16-1)	1.4 (D + F)	N = 1.400; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	11
Equation (16-2a)	1.2 (D + F) + 1.6 (L + H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2b)	1.2 (D + F) + 1.6 (L + H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-2c)	1.2 (D + F) + 1.6 (L + H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3a)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3b)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3c)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + f_1L	N = 1.200; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	9
Equation (16-3d)	1.2 (D + F) + 1.6 (L_r) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-3e)	1.2 (D + F) + 1.6 (S) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-3f)	1.2 (D + F) + 1.6 (R) + 1.6 (H) + (0.5 W)	N = 1.200; V_x = 0.000; V_y = 0.350; M_x = 0.96000; M_y = 0.96000; M_z = 0.00000;	no	33
Equation (16-4a)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (L_r)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-4b)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (S)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-4c)	1.2 (D + F) + 1.0 (W) + f_1L + 1.6 (H) + 0.5 (R)	N = 1.200; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	56
Equation (16-5)	1.2 (D + F) + 1.0 (E) + f_2L + 1.6 (H) + f_2S	N = 1.200; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	62
Equation (16-6)	0.9 (D) + 1.0 (W) + 1.6 (H)	N = 0.900; V_x = 0.000; V_y = 0.700; M_x = 1.92000; M_y = 1.92000; M_z = 0.00000;	no	54
Equation (16-7)	0.9 (D + F) + 1.0 (E) + 1.6 (H)	N = 0.900; V_x = 0.700; V_y = 0.700; M_x = 2.15000; M_y = 2.15000; M_z = 0.00000;	no	59

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

Controlling load case: Equation (16-5) 1.2 (D + F) + 1.0 (E) + f_1L + 1.6 (H) + f_2S



Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.841	0.247	0.175	0.175
2	0.000	0.247	0.175	0.175
3	2.165	0.247	0.175	0.175
4	0.841	0.247	0.175	0.175

Max. concrete compressive strain: 0.07 [‰]
Max. concrete compressive stress: 323 [psi]
Resulting tension force in (x/y)=(-3.095/3.095): 3.847 [kip]
Resulting compression force in (x/y)=(5.247/-5.247): 2.647 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	2.165	20.808	11	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	3.847	6.305	62	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
32.013	0.650	20.808	2.165

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.920	1.262	1.262	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.410	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
230.21	76.74	0.776	0.776	1.000	1.000	5.365

Results

N_{cbg} [kip]	$\phi_{concrete}$	ϕN_{cbg} [kip]	N_{ua} [kip]
9.700	0.650	6.305	3.847

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.247	9.996	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.990	30.043	4	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [kip]
16.660

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.660	0.600	9.996	0.247

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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	2.920	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	4.410	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
306.95	76.74	1.000	1.000	1.000	1.000	5.365

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
42.918	0.700	30.043	0.990

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.610	0.033	5/3	45	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Electrical Room - HSS10x10	Date:	8/17/2026
Fastening point:			

7 Installation data

Profile: Square HSS (AISC), HSS10X10X.375; (L x W x T) = 10.000 in. x 10.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (4)

Item number: 418084 KH-EZ 3/4"x5 1/2"

Maximum installation torque: 0.09501 ft.kip

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 4.375 in.

Minimum thickness of the base material: 6.000 in.

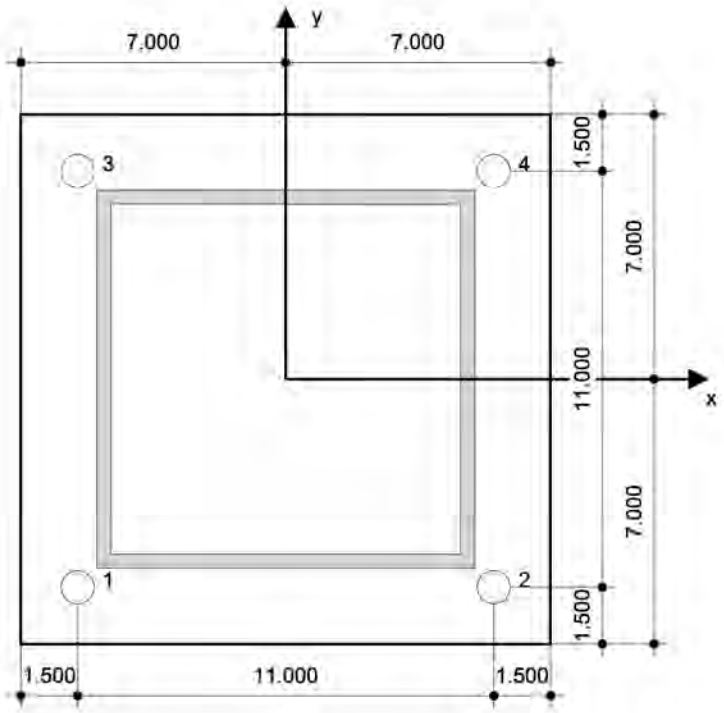
Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 4 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hilti SIW 9-A22 Impact Wrench

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Electrical Room - HSS10x10	Date:	8/17/2026
Fastening point:			



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.500	-5.500	-	-	-	-
2	5.500	-5.500	-	-	-	-
3	-5.500	5.500	-	-	-	-
4	5.500	5.500	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

DESCRIPTION: Electrical Room - Worst Case

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : ACI 318-19
Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	0.950 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

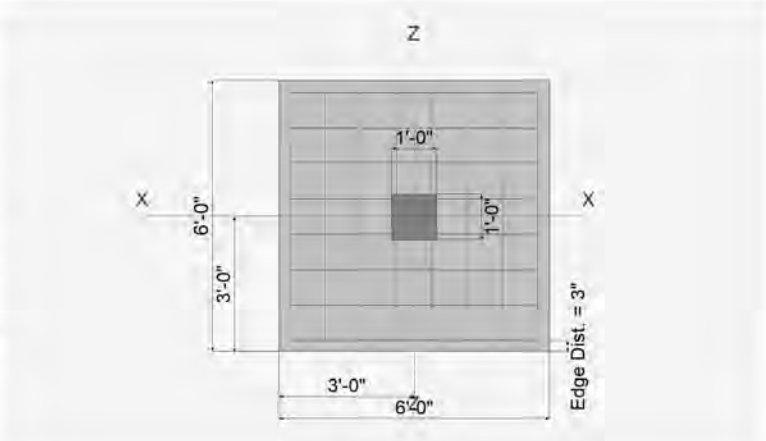
Width parallel to X-X Axis	=	6.0 ft
Length parallel to Z-Z Axis	=	6.0 ft
Footing Thickness	=	12.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	12.0 in
pz : parallel to Z-Z Axis	=	12.0 in
Height	=	1.0 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

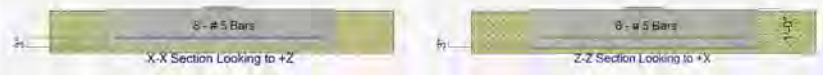
Number of Bars	=	8
Reinforcing Bar Size	=	# 5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	8
Reinforcing Bar Size	=	# 5

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



Top Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=	8
Reinforcing Bar Size	=	5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	8
Reinforcing Bar Size	=	5

Rebar Centerline to Edge of Concrete...

at Top of footing	=	3.0 in

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

DESCRIPTION: Electrical Room - Worst Case

Applied Loads

		D	Lr	L	S	W	E	H
P : Vertical Load	=	0.000	0.000	73.000	0.000	0.000	0.000	0.000 k
OB : Overburden	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 ksf
M-xx	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
M-zz	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
V-x	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k
V-z	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8692	Soil Bearing	2.173 ksf	2.50 ksf	+D+L about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	999.0	Sliding - X-X	0.0 k	2.243 k	D Only
PASS	999.0	Sliding - Z-Z	0.0 k	2.243 k	D Only
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.6268	Z Flexure (+X) Bot Tens	10.138 k-ft/ft	16.175 k-ft/ft	+1.20D+1.60L
PASS	0.6268	Z Flexure (-X) Bot Tens	10.138 k-ft/ft	16.175 k-ft/ft	+1.20D+1.60L
PASS	0.627	Min Steel X Flexure Bottom	0.259 in2/ft	0.413 in2/ft	n/a
PASS	0.6268	X Flexure (+Z) Bot Tens	10.138 k-ft/ft	16.175 k-ft/ft	+1.20D+1.60L
PASS	0.6268	X Flexure (-Z) Bot Tens	10.138 k-ft/ft	16.175 k-ft/ft	+1.20D+1.60L
PASS	0.627	Min Steel Z Flexure Bottom	0.259 in2/ft	0.413 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.8806	1-way Shear (+X)	52.272 psi	59.357 psi	+1.20D+1.60L
PASS	0.8806	1-way Shear (-X)	52.272 psi	59.357 psi	+1.20D+1.60L
PASS	0.8806	1-way Shear (+Z)	52.272 psi	59.357 psi	+1.20D+1.60L
PASS	0.8806	1-way Shear (-Z)	52.272 psi	59.357 psi	+1.20D+1.60L
PASS	0.7410	2-way Punching	140.593 psi	189.737 psi	+1.20D+1.60L

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.1450	0.1450	n/a	n/a	0.058
X-X, +D+L	2.50	n/a	0.0	2.173	2.173	n/a	n/a	0.869
X-X, +D+0.750L	2.50	n/a	0.0	1.666	1.666	n/a	n/a	0.666
X-X, +0.60D	2.50	n/a	0.0	0.0870	0.0870	n/a	n/a	0.035
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.1450	0.1450	0.058
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	2.173	2.173	0.869
Z-Z, +D+0.750L	2.50	0.0	n/a	n/a	n/a	1.666	1.666	0.666
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.0870	0.0870	0.035

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft) Overturning	Resisting	Ratio	Status	Moment Sum About Positive Edge (k-ft) Overturning	Resisting	Ratio	Status
Footing Has NO Overturning								

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: bldg 1.ec6

LIC# : KW-06018138, Build:20.26.05.05 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Electrical Room - Worst Case

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.40D	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D+1.60L	10.138	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D+1.60L	10.138	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D+0.50L	3.168	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D+0.50L	3.168	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +1.20D	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +0.90D	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
X-X, +0.90D	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.40D	0.0	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.40D	0.0	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D+1.60L	10.138	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D+1.60L	10.138	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D+0.50L	3.168	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D+0.50L	3.168	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D	0.0	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +1.20D	0.0	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +0.90D	0.0	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK
Z-Z, +0.90D	0.0	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	16.175	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK
+1.20D+1.60L	52.27 psi	52.27 psi	52.27 psi	59.36 psi	0.88	OK
+1.20D+0.50L	16.34 psi	16.34 psi	16.34 psi	59.36 psi	0.28	OK
+1.20D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK

One Way Shear Z

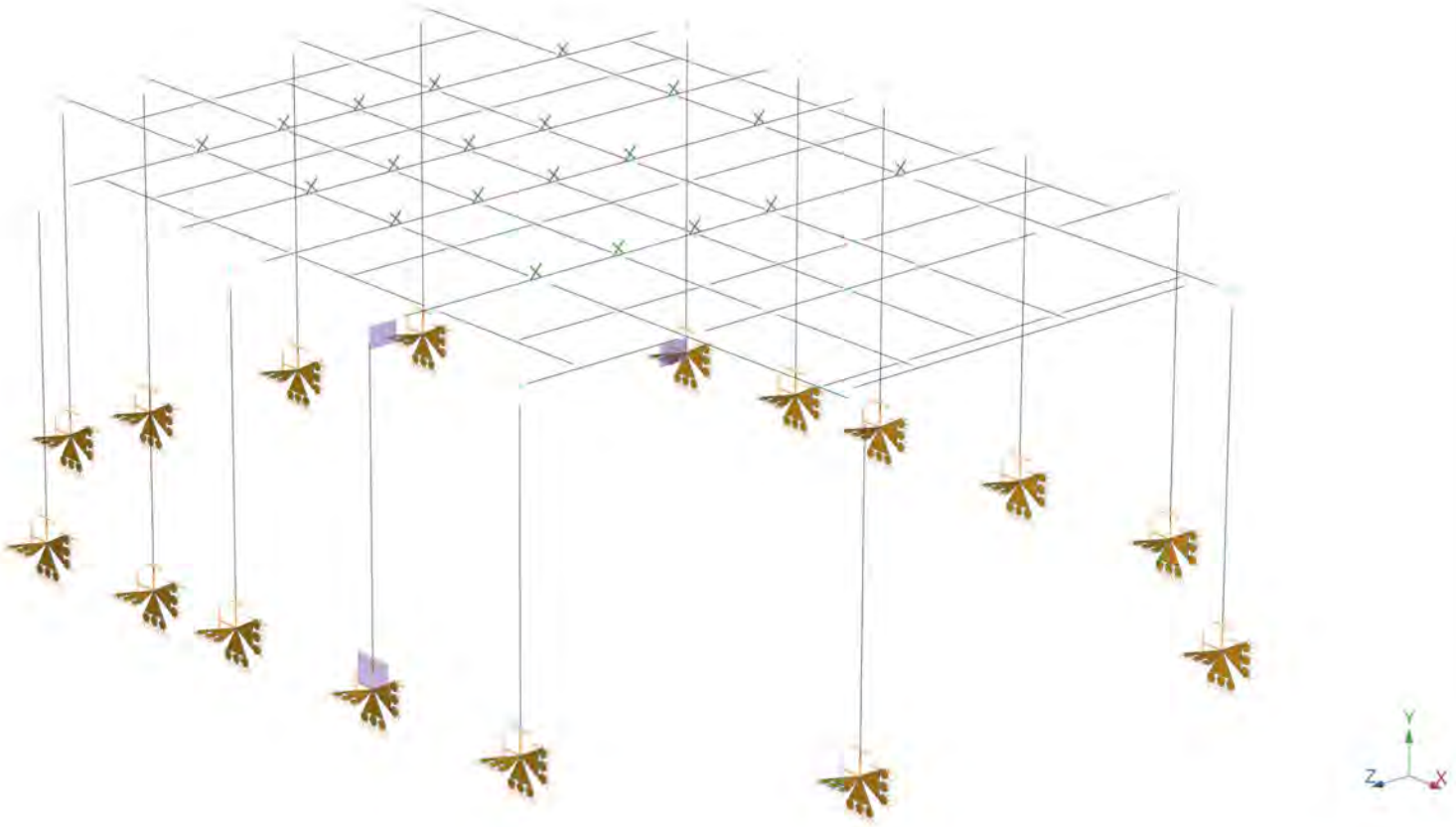
Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK
+1.20D+1.60L	52.27 psi	52.27 psi	52.27 psi	59.36 psi	0.88	OK
+1.20D+0.50L	16.34 psi	16.34 psi	16.34 psi	59.36 psi	0.28	OK
+1.20D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	59.36 psi	0.00	OK

Two-Way "Punching" Shear

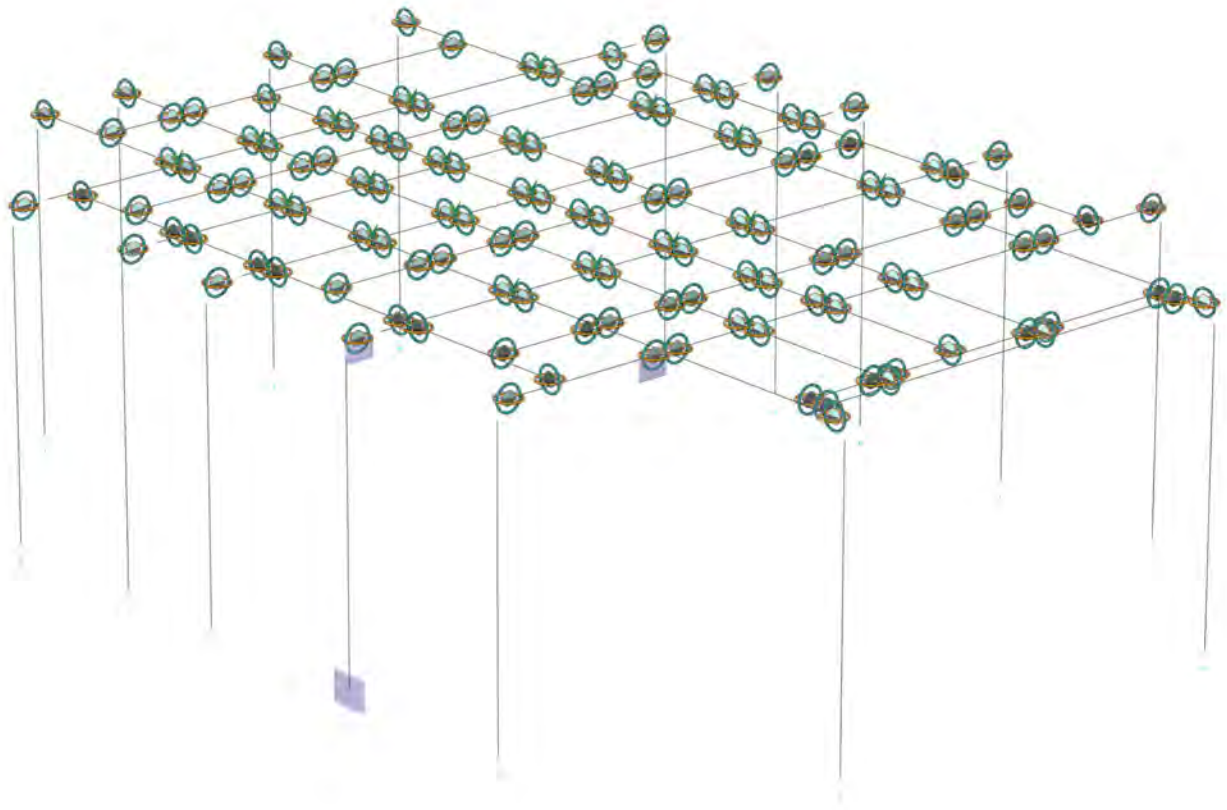
All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	189.74 psi	0.00	OK
+1.20D+1.60L	140.59 psi	189.74 psi	0.74	OK
+1.20D+0.50L	43.94 psi	189.74 psi	0.23	OK
+1.20D	0.00 psi	189.74 psi	0.00	OK
+0.90D	0.00 psi	189.74 psi	0.00	OK

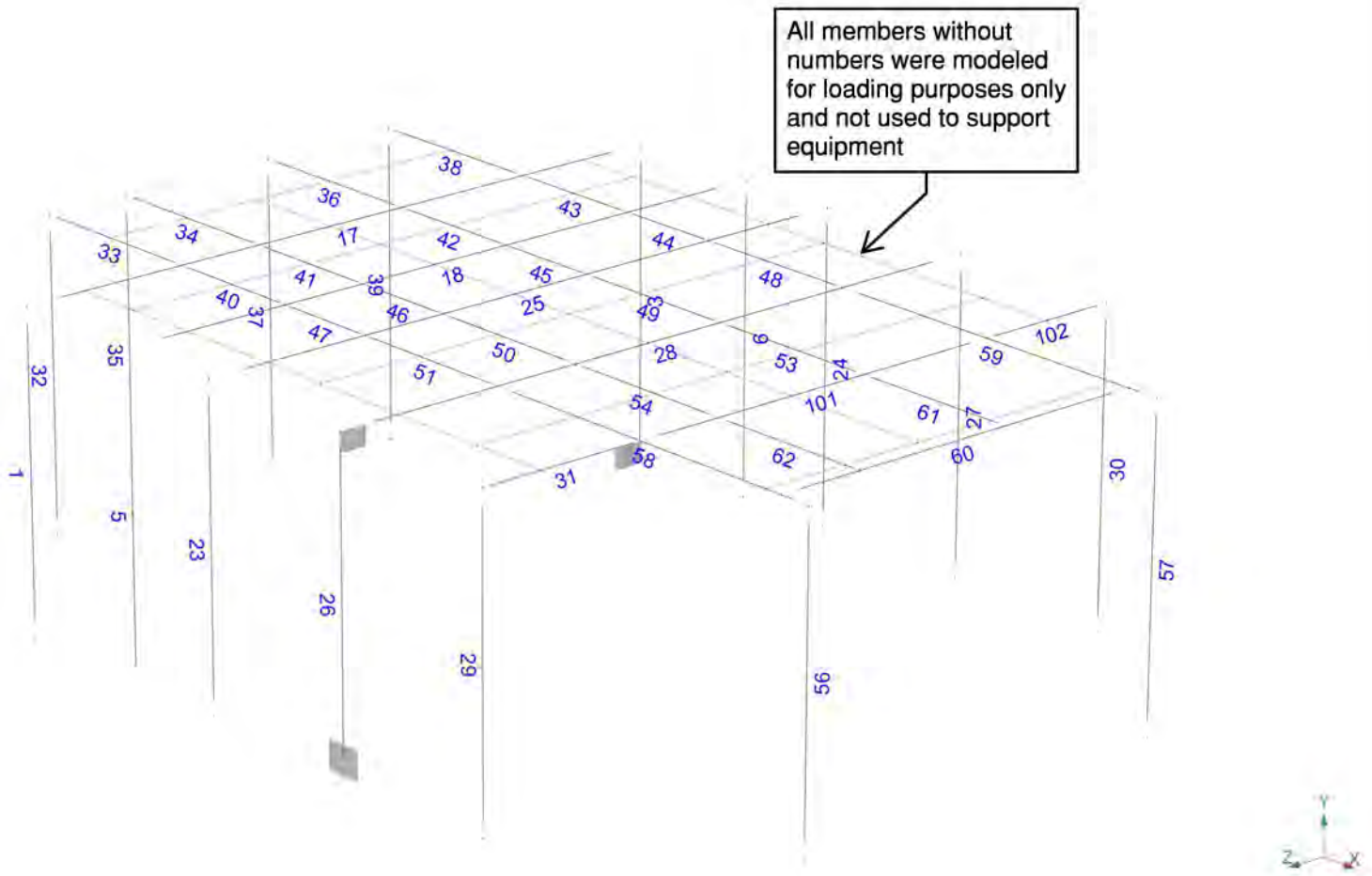
Cable Tray Equipment Support Frames
Near Grids F and 1
Column Supports



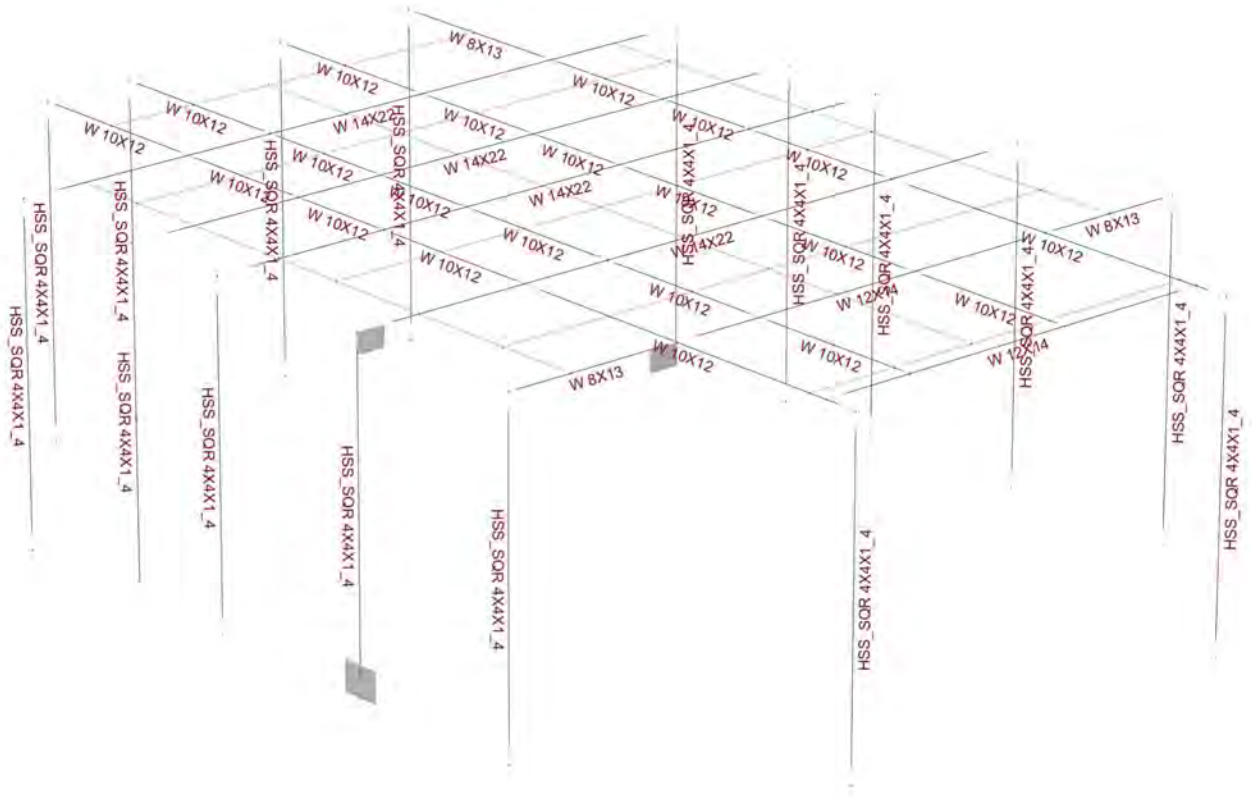
Cable Tray Equipment Support Frames
Near Grids F and 1
Beam Hinges

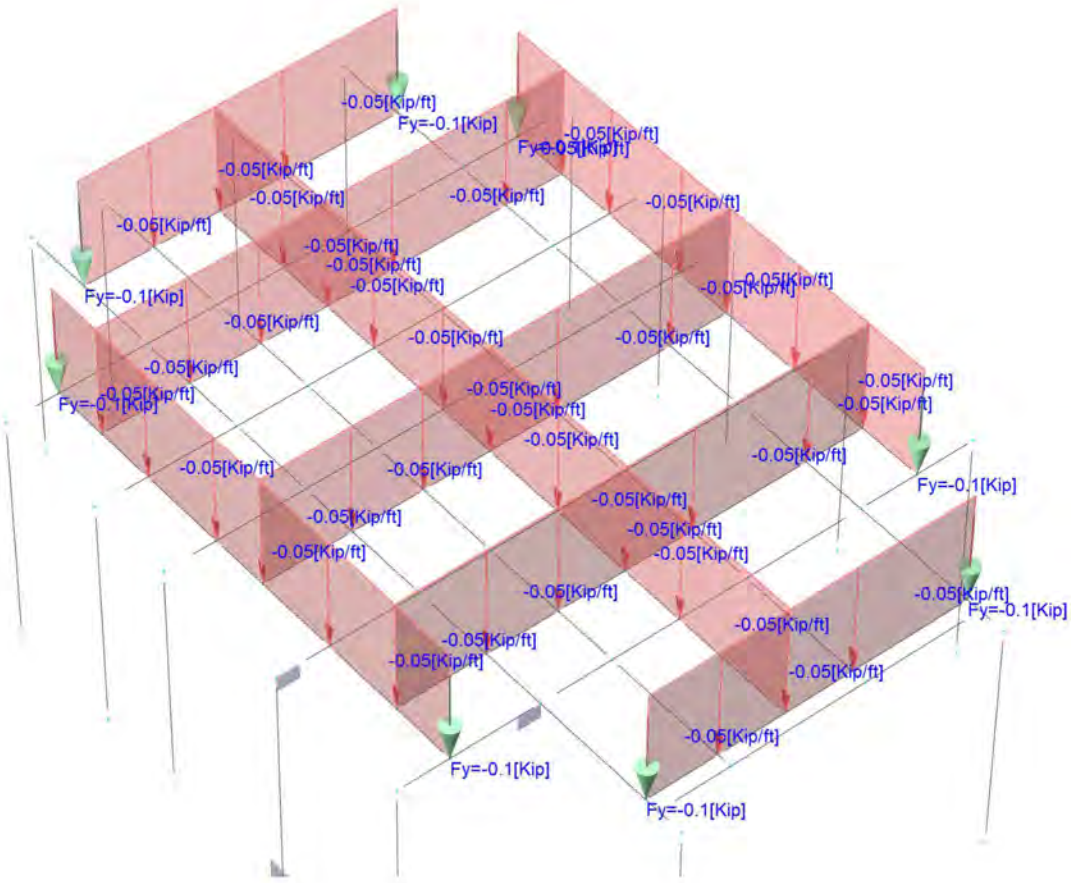


Cable Tray Equipment Support Frames
Near Grids F and 1
Member Numbers



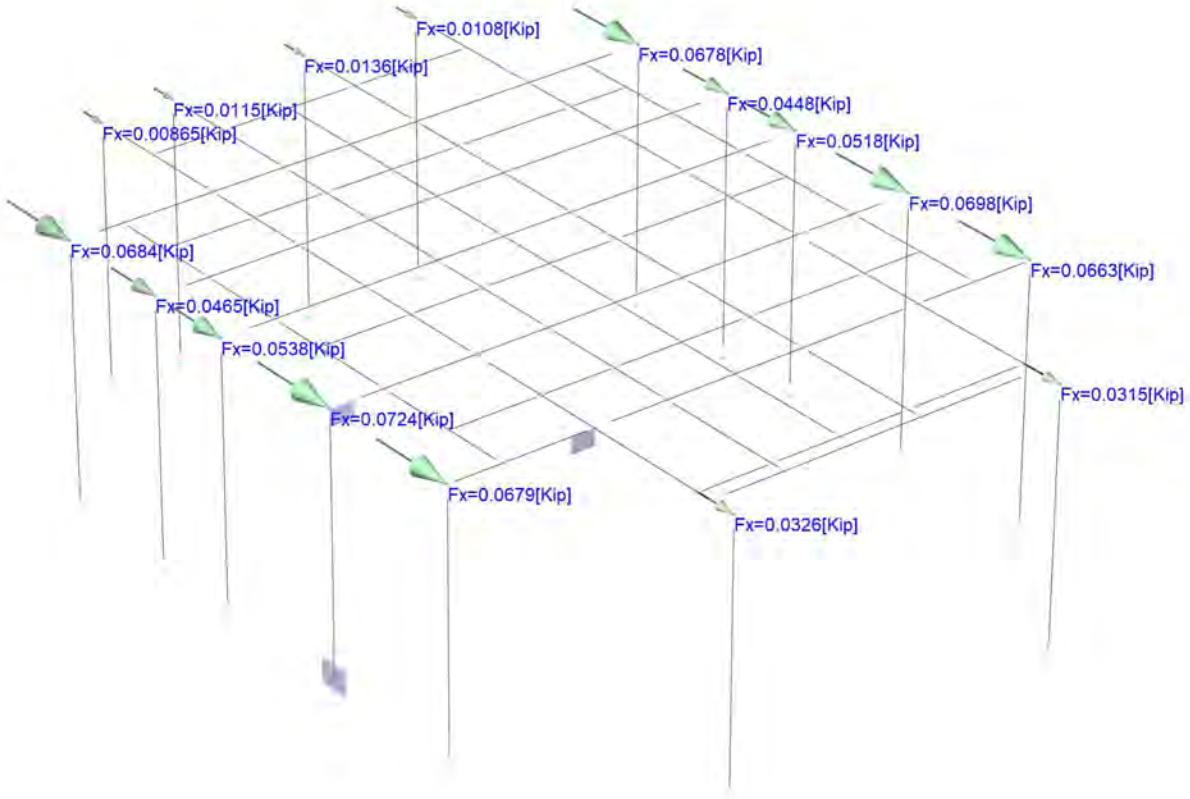
Cable Tray Equipment Support Frames
Near Grids F and 1
Member Numbers





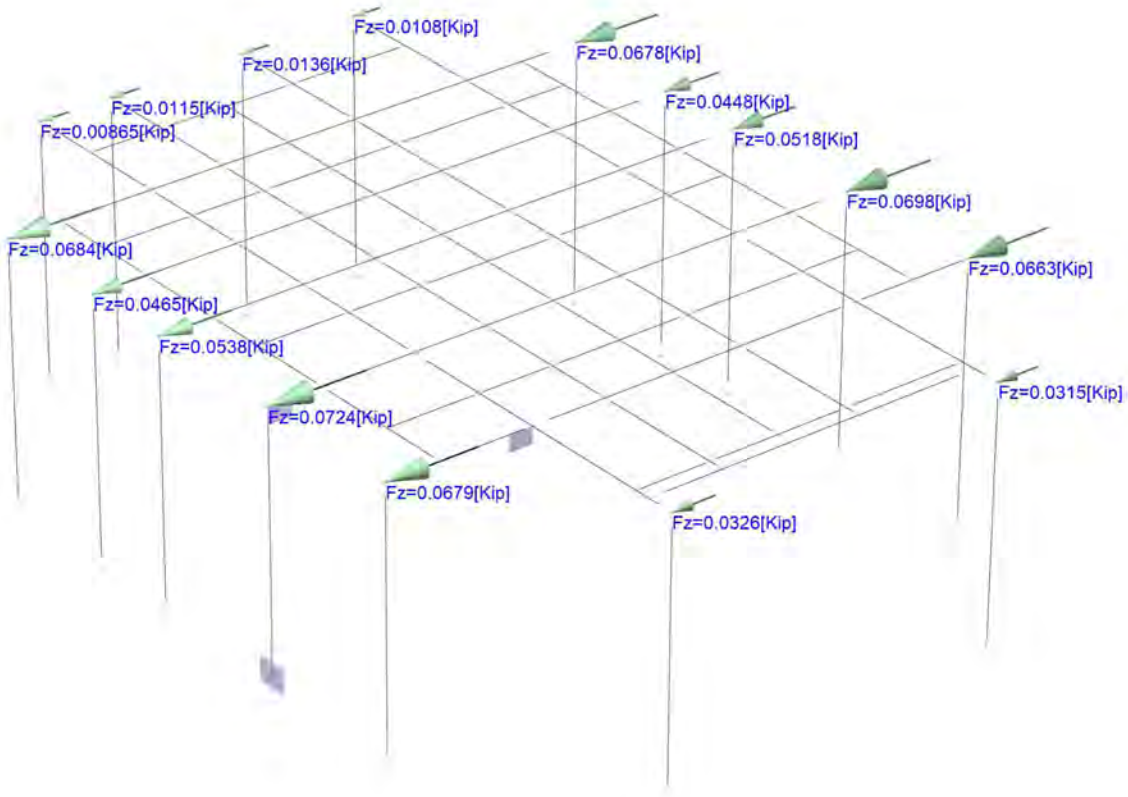
Cable Tray Equipment Support Frames Near Grids F and 1 Equipment Loads

Cable Tray Equipment Support Frames Near Grids F and 1 Lateral Loads in X Direction



Loads
Concentrated - Nodes

Cable Tray Equipment Support Frames Near Grids F and 1 Lateral Loads in Z Direction



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:33 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Technology Cable Tray Supports in Office Area\Cable Tray Support in MMR Room Near Grids F and 1.retx

Steel Code Check
Summary - Group by description

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+E1
D4=1.2DL+E2
D5=1.2DL+LL+E1
D6=1.2DL+LL+E2
D7=0.9DL+E1
D8=0.9DL+E2

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam to beam	*W 10X12*	53	D2 at 55.83%	0.01	OK	
Column at girder	*HSS_SQR 4X4X1_4*	26	D6 at 0.00%	0.04	OK	
Girder	*W 14X22*	28	D2 at 47.78%	0.06	OK	
Girder 2	*W 8X13*	31	D2 at 36.84%	0.01	OK	
Girder 3	*W 12X14*	101	D2 at 42.61%	**0.06**	**OK**	
North beam to beam		60	D2 at 36.36%	0.05	OK	
North Column	*HSS_SQR 4X4X1_4*	57	D5 at 0.00%	0.02	OK	
South beam to column	*W 10X12*	36	D2 at 45.88%	0.01	OK	
South Column	*HSS_SQR 4X4X1_4*	39	D5 at 0.00%	0.02	OK	

RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:34 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Technology Cable Tray Supports in Office Area\Cable Tray Support in MMR Room Near Grids F and 1.retx

Steel Code Check
Comprehensive Report

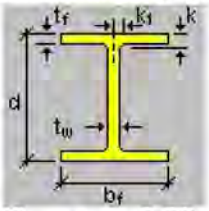
AISC 360-2016 LRFD (Hot-rolled)

Member : 53 (Beam to beam) - OK

Section information

Section name: W 10X12 **(US)**

Dimensions



bf	=	3.960	[in]	Width
d	=	9.870	[in]	Depth
k	=	0.510	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.210	[in]	Flange thickness
tw	=	0.190	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.540	
Moment of Inertia (local axes) (I)	[in4]	53.800	2.180
Moment of Inertia (principal axes) (I')	[in4]	53.800	2.180
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.898	0.785
Radius of gyration (principal axes) (r')	[in]	3.898	0.785
Saint-Venant torsion constant. (J)	[in4]	0.055	
Section warping constant. (Cw)	[in6]	50.900	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	10.900	1.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	10.900	1.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	10.900	1.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	10.900	1.100
Plastic section modulus (local axis) (Z)	[in3]	12.600	1.740
Plastic section modulus (principal axis) (Z')	[in3]	12.600	1.740
Polar radius of gyration. (ro)	[in]	3.977	
Area for shear (Aw)	[in2]	1.660	1.880
Torsional constant. (C)	[in3]	0.222	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	5.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
5.00	5.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
5.00	5.00	5.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	159.30 [Kip]	Reference	:	Cl.D2
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D5 at 55.83%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	159.30	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	177.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	143.95 [Kip]	Reference	:	Cl.E3
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	9.43	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	46.58	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	143.95	Cl.E3
Effective length (KL_{33})	[ft]	5.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	15.39	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	1208.30	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.25	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	49.14	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	159.95	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	102.29 [Kip]	Reference	:	Cl.E3
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	9.43	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	46.58	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	102.29	Cl.E3
Effective length (KL_{22})	[ft]	5.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	76.46	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	48.96	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.49	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	32.61	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	113.65	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	118.43	Cl.E4
Effective length (KL_{11})	[ft]	5.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	83.19	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	3.38	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	38.88	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	131.59	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.01			
Capacity	:	46.90 [Kip*ft]	Reference	:	Cl.F3.1
Demand	:	0.55 [Kip*ft]	Ctrl Eq.	:	D2 at 55.83%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Noncompact	
Unstiffened element slenderness (λ)	--	9.43	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	46.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	47.25	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	2.77	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	0.98	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	8.05	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.37	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	52.50	Eq.F2-2
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	46.90	Cl.F3.1
Flange local buckling (M_n)	[Kip*ft]	52.12	Eq.F3-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	6.46 [Kip*ft]	Reference	:	Cl.F6.2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D4 at 55.83%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Noncompact	
Unstiffened element slenderness (λ)	--	9.43	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	46.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	6.53	Cl.F6.1
Yielding (M_n)	[Kip*ft]	7.25	Eq.F6-1
Factored compression flange local buckling strength about a geometric ...	[Kip*ft]	6.46	Cl.F6.2
Flange local buckling (M_n)	[Kip*ft]	7.18	Eq.F6-2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	44.91 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D4 at 55.83%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	44.91	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	49.90	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	56.40 [Kip]	Reference	:	Cl.G1
Demand	:	0.25 [Kip]	Ctrl Eq.	:	D2 at 55.83%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	56.40	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	56.40	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.01			
Ctrl Eq.	:	D2 at 55.83%	Reference	:	Eq.H1-1b

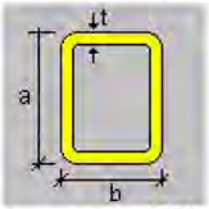
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.01	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	46.90	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	6.46	Cl.H1.1
Available axial strength (P_c)	[Kip]	102.29	Cl.H1.1

Member : 26 (Column at girder) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	

Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	8.50

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
8.50	8.50

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
8.50	8.50	8.50	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.00			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	: 0.01			
Capacity	: 109.17 [Kip]	Reference	: Cl.E3	
Demand	: 1.59 [Kip]	Ctrl Eq.	: D2 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	14.17	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	109.17	Cl.E3
Effective length (KL_{33})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	67.05	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	121.30	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.01			
Capacity	: 109.17 [Kip]	Reference	: Cl.E3	
Demand	: 1.59 [Kip]	Ctrl Eq.	: D2 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	14.17	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	109.17	Cl.E3
Effective length (KL_{22})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	67.05	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	121.30	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.02			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-0.43 [Kip*ft]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	–	58.28	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	33.72	
Limiting slenderness for compact stiffened element (λ_p)	–	26.97	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.04			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.65 [Kip*ft]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	–	58.28	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	33.72	
Limiting slenderness for compact stiffened element (λ_p)	–	26.97	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.08 [Kip]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.05 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.04			
Ctrl Eq.	:	D6 at 0.00%	Reference	:	Eq.H1-1b

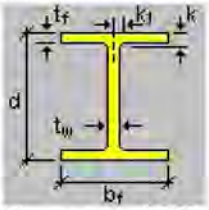
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.04	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	109.17	Cl.H1.1

Member : 28 (Girder) - OK

Section information

Section name: W 14X22 **(US)**

Dimensions



bf	=	5.000	[in]	Width
d	=	13.700	[in]	Depth
k	=	0.735	[in]	Distance k
k1	=	0.750	[in]	Distance k1
tf	=	0.335	[in]	Flange thickness
tw	=	0.230	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	6.490	
Moment of Inertia (local axes) (I)	[in4]	199.000	7.000
Moment of Inertia (principal axes) (I')	[in4]	199.000	7.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.537	1.039
Radius of gyration (principal axes) (r')	[in]	5.537	1.039
Saint-Venant torsion constant. (J)	[in4]	0.208	
Section warping constant. (Cw)	[in6]	314.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	29.000	2.800
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	29.000	2.800
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	29.000	2.800
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	29.000	2.800
Plastic section modulus (local axis) (Z)	[in3]	33.200	4.390
Plastic section modulus (principal axis) (Z')	[in3]	33.200	4.390
Polar radius of gyration. (ro)	[in]	5.634	
Area for shear (Aw)	[in2]	3.350	3.150
Torsional constant. (C)	[in3]	0.536	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	19.75

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
4.75	19.75
2.42	
2.27	
2.31	
4.00	
4.00	

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
19.75	4.75	19.75	1.0	1.0	1.0
	2.42			1.0	
	2.27			1.0	
	2.31			1.0	
	4.00			1.0	
	4.00			1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	292.05 [Kip]	Reference	:	Cl.D2
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D4 at 89.87%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	292.05	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	324.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	231.46 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.46	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	53.17	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	231.46	Cl.E3
Effective length (KL_{33})	[ft]	19.75	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	42.80	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	156.25	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	5.88	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	43.73	Eq.E3-2

Nominal flexural buckling strength (P_{n33})	[Kip]	257.18	Eq.E7-1

Compression in the minor axis 22

Ratio	: 0.00		
Capacity	: 97.48 [Kip]	Reference	: Cl.E4
Demand	: 0.00 [Kip]	Ctrl Eq.	: D4 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	7.46	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	53.17	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	215.08	Cl.E3
Effective length (KL_{22})	[ft]	4.75	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	54.88	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	95.02	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	5.96	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	40.12	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	238.98	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	97.48	Cl.E4
Effective length (KL_{11})	[ft]	19.75	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	19.03	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	6.49	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	16.69	Eq.E3-3
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	108.31	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	: 0.06		
Capacity	: 124.50 [Kip*ft]	Reference	: Cl.F2.1
Demand	: 7.46 [Kip*ft]	Ctrl Eq.	: D2 at 47.78%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	7.46	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	–	9.15	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	53.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	124.50	Cl.F2.1
Yielding (M_n)	[Kip*ft]	138.33	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	124.50	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	3.67	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.27	Eq.F2-7
Lateral-torsional factor (c)	–	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	10.44	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	–	1.12	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	138.33	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	16.46 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.07 [Kip*ft]	Ctrl Eq.	:	D5 at 36.29%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.46	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	53.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	16.46	Cl.F6.1
Yielding (M_n)	[Kip*ft]	18.29	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	90.45 [Kip]	Reference	:	Cl.G1
Demand	:	0.02 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	90.45	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	100.50	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.02			
Capacity	:	94.50 [Kip]	Reference	:	Cl.G1
Demand	:	1.59 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	94.50	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	94.50	Eq.G2-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.06			
Ctrl Eq.	:	D2 at 47.78%	Reference	:	Eq.H1-1b

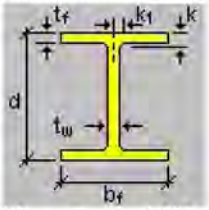
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.06	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	124.50	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	16.46	Cl.H1.1
Available axial strength (P_c)	[Kip]	292.05	Cl.H1.1

Member : 31 (Girder 2) - OK

Section information

Section name: W 8X13 **(US)**

Dimensions



bf	=	4.000	[in]	Width
d	=	7.990	[in]	Depth
k	=	0.555	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.255	[in]	Flange thickness
tw	=	0.230	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.840	
Moment of Inertia (local axes) (I)	[in4]	39.600	2.730
Moment of Inertia (principal axes) (I')	[in4]	39.600	2.730
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.211	0.843
Radius of gyration (principal axes) (r')	[in]	3.211	0.843
Saint-Venant torsion constant. (J)	[in4]	0.087	
Section warping constant. (Cw)	[in6]	40.800	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	9.910	1.370
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	9.910	1.370
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	9.910	1.370
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	9.910	1.370
Plastic section modulus (local axis) (Z)	[in3]	11.400	2.150
Plastic section modulus (principal axis) (Z')	[in3]	11.400	2.150
Polar radius of gyration. (ro)	[in]	3.320	
Area for shear (Aw)	[in2]	2.040	1.840
Torsional constant. (C)	[in3]	0.296	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	4.75

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
4.75	4.75

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
4.75	4.75	4.75	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	172.80 [Kip]	Reference	:	Cl.D2
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D4 at 36.84%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	172.80	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	192.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	168.86 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D2 at 36.84%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.84	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	29.91	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	168.86	Cl.E3
Effective length (KL_{33})	[ft]	4.75	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	17.75	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	908.47	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.84	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	48.86	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	187.63	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	123.72 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D2 at 36.84%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.84	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	29.91	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	123.72	Cl.E3
Effective length (KL_{22})	[ft]	4.75	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	67.60	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	62.63	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.84	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	35.80	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	137.46	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	142.32	Cl.E4
Effective length (KL_{11})	[ft]	4.75	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	107.86	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	3.84	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	41.18	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	158.14	Eq.E4-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.01			
Capacity	:	42.75 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	0.47 [Kip*ft]	Ctrl Eq.	:	D2 at 36.84%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.84	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	29.91	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	42.75	Cl.F2.1
Yielding (M_n)	[Kip*ft]	47.50	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	42.75	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	2.98	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.03	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	9.29	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.41	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	47.50	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	8.06 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.03 [Kip*ft]	Ctrl Eq.	:	D3 at 36.84%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.84	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	29.91	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	8.06	Cl.F6.1
Yielding (M_n)	[Kip*ft]	8.96	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	55.08 [Kip]	Reference	:	Cl.G1
Demand	:	0.02 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	55.08	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	61.20	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	55.20 [Kip]	Reference	:	Cl.G1
Demand	:	0.27 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	55.20	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	55.20	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.01			
Ctrl Eq.	:	D2 at 36.84%	Reference	:	Eq.H1-3a(H1-1b)

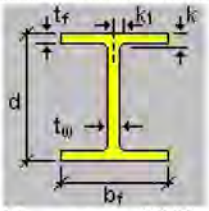
Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending:	--	0.01	Eq.H1-3a(H1-1b)
In-plane available flexural strength (M_{c33})	[Kip*ft]	42.75	Cl.H1.3
In-plane available axial compressive strength (P_c)	[Kip]	168.86	Cl.H1.3
Interaction for doubly symmetric members for out-of-plane bending:	--	0.00	Eq.H1-3b
Out-of-plane available flexural-torsional strength (M_{c33})	[Kip*ft]	38.05	Cl.H1.3
Out-of-plane available axial compressive strength (P_{co})	[Kip]	123.72	Cl.H1.3

Member : 101 (Girder 3) - OK

Section information

Section name: W 12X14 **(US)**

Dimensions



bf	=	3.970	[in]	Width
d	=	11.900	[in]	Depth
k	=	0.525	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.225	[in]	Flange thickness
tw	=	0.200	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.160	
Moment of Inertia (local axes) (I)	[in4]	88.600	2.360
Moment of Inertia (principal axes) (I')	[in4]	88.600	2.360
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.615	0.753
Radius of gyration (principal axes) (r')	[in]	4.615	0.753
Saint-Venant torsion constant. (J)	[in4]	0.070	
Section warping constant. (Cw)	[in6]	80.400	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.900	1.190
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.900	1.190
Plastic section modulus (local axis) (Z)	[in3]	17.400	1.900
Plastic section modulus (principal axis) (Z')	[in3]	17.400	1.900
Polar radius of gyration. (ro)	[in]	4.676	
Area for shear (Aw)	[in2]	1.790	2.380
Torsional constant. (C)	[in3]	0.272	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	11.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.00	11.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
11.00	11.00	11.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 187.20 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D4 at 63.64%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	187.20	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	208.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 153.19 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D4 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	8.82	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	54.25	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	153.19	Cl.E3
Effective length (KL_{33})	[ft]	11.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	28.60	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	349.86	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.61	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	47.10	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	170.21	Eq.E7-1

Compression in the minor axis 22

Ratio	: 0.00			
Capacity	: 30.60 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D4 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	8.82	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	54.25	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	30.60	Cl.E3
Effective length (KL_{22})	[ft]	11.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	175.25	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	9.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	4.16	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	8.17	Eq.E3-3

Nominal flexural buckling strength (P_{n22})	[Kip]	34.00	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	75.81	Cl.E4
Effective length (KL_{11})	[ft]	11.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	23.15	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	4.16	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	20.25	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	84.24	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.06			
Capacity	:	25.50 [Kip*ft]	Reference	:	Cl.F2.2
Demand	:	1.54 [Kip*ft]	Ctrl Eq.	:	D2 at 42.61%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.82	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	54.25	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.25	Cl.F2.1
Yielding (M_n)	[Kip*ft]	72.50	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	25.50	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	2.66	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	0.96	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	7.73	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.19	Eq.F1-1
Critical stress (F_{cr})	[Kip/in2]	22.82	Eq.F2-4
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	28.33	Eq.F2-3

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	7.13 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.03 [Kip*ft]	Ctrl Eq.	:	D5 at 21.97%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.82	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	54.25	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	7.13	Cl.F6.1
Yielding (M_n)	[Kip*ft]	7.92	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	48.24 [Kip]	Reference	: Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	: D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	48.24	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	53.59	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.01		
Capacity	:	64.26 [Kip]	Reference	: Cl.G1
Demand	:	0.45 [Kip]	Ctrl Eq.	: D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	64.26	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	71.40	Eq.G2-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.06		
Ctrl Eq.	:	D2 at 42.61%	Reference	: Eq.H1-1b

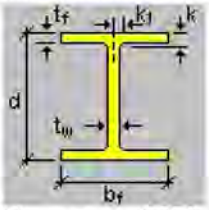
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.06	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	25.50	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	7.13	Cl.H1.1
Available axial strength (P_c)	[Kip]	187.20	Cl.H1.1

Member : 60 (North beam to beam) - OK

Section information

Section name: W 12X14 **(US)**

Dimensions



bf	=	3.970	[in]	Width
d	=	11.900	[in]	Depth
k	=	0.525	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.225	[in]	Flange thickness
tw	=	0.200	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.160	
Moment of Inertia (local axes) (I)	[in4]	88.600	2.360
Moment of Inertia (principal axes) (I')	[in4]	88.600	2.360
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.615	0.753
Radius of gyration (principal axes) (r')	[in]	4.615	0.753
Saint-Venant torsion constant. (J)	[in4]	0.070	
Section warping constant. (Cw)	[in6]	80.400	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.900	1.190
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.900	1.190
Plastic section modulus (local axis) (Z)	[in3]	17.400	1.900
Plastic section modulus (principal axis) (Z')	[in3]	17.400	1.900
Polar radius of gyration. (ro)	[in]	4.676	
Area for shear (Aw)	[in2]	1.790	2.380
Torsional constant. (C)	[in3]	0.272	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	11.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.00	11.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
11.00	11.00	11.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	187.20 [Kip]	Reference	:	Cl.D2
Demand	:	0.02 [Kip]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	187.20	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	208.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	153.19 [Kip]	Reference	:	Cl.E3
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D4 at 78.03%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	8.82	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	54.25	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	153.19	Cl.E3
Effective length (KL_{33})	[ft]	11.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	28.60	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	349.86	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.61	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	47.10	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	170.21	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	30.60 [Kip]	Reference	:	Cl.E3
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D4 at 78.03%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	8.82	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	54.25	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	30.60	Cl.E3
Effective length (KL_{22})	[ft]	11.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	175.25	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	9.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	4.16	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	8.17	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	34.00	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	75.81	Cl.E4
Effective length (KL_{11})	[ft]	11.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	23.15	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	4.16	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	20.25	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	84.24	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.05			
Capacity	:	25.36 [Kip*ft]	Reference	:	Cl.F2.2
Demand	:	1.26 [Kip*ft]	Ctrl Eq.	:	D2 at 36.36%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.82	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	54.25	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.25	Cl.F2.1
Yielding (M_n)	[Kip*ft]	72.50	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	25.36	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	2.66	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	0.96	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	7.73	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.18	Eq.F1-1
Critical stress (F_{cr})	[Kip/in2]	22.70	Eq.F2-4
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	28.18	Eq.F2-3

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	7.13 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	-0.02 [Kip*ft]	Ctrl Eq.	:	D5 at 36.36%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.82	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	54.25	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	7.13	Cl.F6.1
Yielding (M_n)	[Kip*ft]	7.92	Eq.F6-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	48.24 [Kip]	Reference	:	Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D5 at 78.03%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	48.24	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	53.59	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	64.26 [Kip]	Reference	:	Cl.G1
Demand	:	0.39 [Kip]	Ctrl Eq.	:	D2 at 78.03%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	64.26	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	71.40	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.03			
Ctrl Eq.	:	D6 at 36.36%	Reference	:	Eq.H1-1b

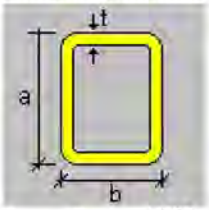
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.03	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	25.37	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	7.13	Cl.H1.1
Available axial strength (P_c)	[Kip]	187.20	Cl.H1.1

Member : 57 (North Column) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	8.50

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
8.50	8.50

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
8.50	8.50	8.50	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	109.17 [Kip]	Reference	:	Cl.E3
Demand	:	0.89 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	

Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	109.17	Cl.E3
Effective length (KL_{33})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	67.05	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	121.30	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.01		
Capacity	:	109.17 [Kip]	Reference	: Cl.E3
Demand	:	0.89 [Kip]	Ctrl Eq.	: D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	14.17	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	109.17	Cl.E3
Effective length (KL_{22})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	67.05	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	121.30	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.02		
Capacity	:	17.59 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-0.34 [Kip*ft]	Ctrl Eq.	: D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.02			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.30 [Kip*ft]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.04 [Kip]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.04 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio : 0.02
Ctrl Eq. : D5 at 0.00% Reference : Eq.H1-1b

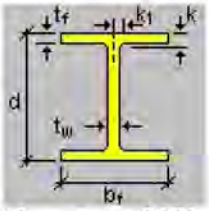
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.02	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	109.17	Cl.H1.1

Member : 36 (South beam to column) - OK

Section information

Section name: W 10X12 **(US)**

Dimensions



bf	=	3.960	[in]	Width
d	=	9.870	[in]	Depth
k	=	0.510	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.210	[in]	Flange thickness
tw	=	0.190	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.540	
Moment of Inertia (local axes) (I)	[in4]	53.800	2.180
Moment of Inertia (principal axes) (I')	[in4]	53.800	2.180
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.898	0.785
Radius of gyration (principal axes) (r')	[in]	3.898	0.785
Saint-Venant torsion constant. (J)	[in4]	0.055	
Section warping constant. (Cw)	[in6]	50.900	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	10.900	1.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	10.900	1.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	10.900	1.100

Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	10.900	1.100
Plastic section modulus (local axis) (Z)	[in3]	12.600	1.740
Plastic section modulus (principal axis) (Z')	[in3]	12.600	1.740
Polar radius of gyration. (ro)	[in]	3.977	
Area for shear (Aw)	[in2]	1.660	1.880
Torsional constant. (C)	[in3]	0.222	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	5.31

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
5.31	5.31

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
5.31	5.31	5.31	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.00			
Capacity	:	159.30 [Kip]	Reference	:	Cl.D2
Demand	:	0.04 [Kip]	Ctrl Eq.	:	D5 at 45.88%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	159.30	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	177.00	Eq.D2-1

Axial Compression Design

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	143.69 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D2 at 45.88%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.43	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	46.58	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	143.69	Cl.E3
Effective length (KL_{33})	[ft]	5.31	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	16.35	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	1070.33	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.26	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	49.03	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	159.65	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	97.69 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D2 at 45.88%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.43	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	46.58	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	97.69	Cl.E3
Effective length (KL_{22})	[ft]	5.31	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	81.24	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	43.37	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.52	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	30.86	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	108.54	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	115.73	Cl.E4
Effective length (KL_{11})	[ft]	5.31	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	74.93	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	3.40	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	37.82	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	128.59	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.01			
Capacity	:	46.90 [Kip*ft]	Reference	:	Cl.F3.1
Demand	:	0.53 [Kip*ft]	Ctrl Eq.	:	D2 at 45.88%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Noncompact	
Unstiffened element slenderness (λ)	–	9.43	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	–	9.15	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	46.58	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	90.55	
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	47.25	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	2.77	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	0.98	Eq.F2-7
Lateral-torsional factor (c)	–	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	8.05	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	–	1.36	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	52.50	Eq.F2-2
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	46.90	Cl.F3.1
Flange local buckling (M_n)	[Kip*ft]	52.12	Eq.F3-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	6.46 [Kip*ft]	Reference	:	Cl.F6.2
Demand	:	0.01 [Kip*ft]	Ctrl Eq.	:	D6 at 45.88%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Noncompact	
Unstiffened element slenderness (λ)	–	9.43	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	–	9.15	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	46.58	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	6.53	Cl.F6.1
Yielding (M_n)	[Kip*ft]	7.25	Eq.F6-1
Factored compression flange local buckling strength about a geometric ...	[Kip*ft]	6.46	Cl.F6.2
Flange local buckling (M_n)	[Kip*ft]	7.18	Eq.F6-2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	44.91 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	44.91	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	49.90	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	56.40 [Kip]	Reference	:	Cl.G1
Demand	:	0.22 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	56.40	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	56.40	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.01			
Ctrl Eq.	:	D2 at 45.88%	Reference	:	Eq.H1-1b

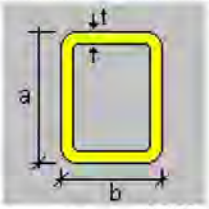
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.01	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	46.90	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	6.46	Cl.H1.1
Available axial strength (P_c)	[Kip]	97.69	Cl.H1.1

Member : 39 (South Column) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	8.50

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
8.50	8.50

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
8.50	8.50	8.50	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 151.65 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 109.17 [Kip]	Reference	: Cl.E3	
Demand	: 0.17 [Kip]	Ctrl Eq.	: D2 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	109.17	Cl.E3
Effective length (KL_{33})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	67.05	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	121.30	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	109.17 [Kip]	Reference	:	Cl.E3
Demand	:	0.17 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	109.17	Cl.E3
Effective length (KL_{22})	[ft]	8.50	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	67.05	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	63.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	35.99	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	121.30	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.02			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-0.34 [Kip*ft]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.01			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.09 [Kip*ft]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	

Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	41.53 [Kip]	Reference	: Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	: D4 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.00		
Capacity	:	41.53 [Kip]	Reference	: Cl.G1
Demand	:	0.04 [Kip]	Ctrl Eq.	: D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00		
Capacity	:	14.77 [Kip*ft]	Reference	: Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.02		
Ctrl Eq.	:	D5 at 0.00%	Reference	: Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.02	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	109.17	Cl.H1.1

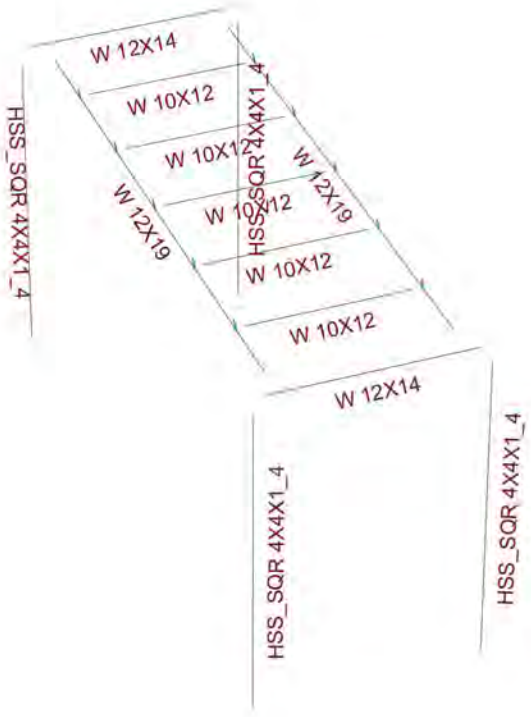
Cable Tray Supports in Hall Near Grids E and 2
3D Rendering



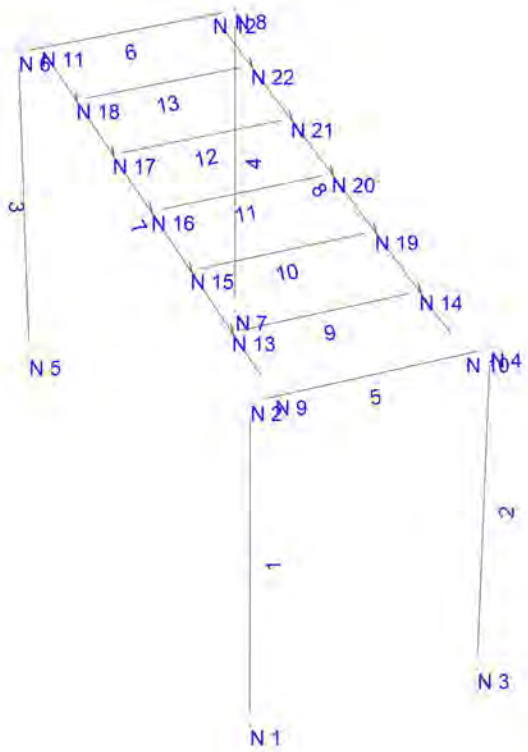
Cable Tray Supports in Hall Near Grids E and 2
Column Supports and Beam Hinges



Cable Tray Supports in Hall Near Grids E and 2
Member Sizes

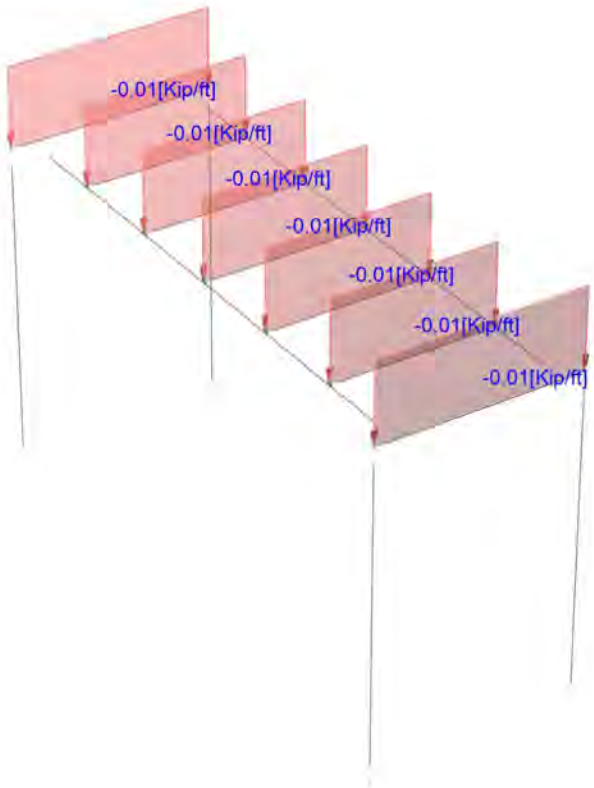


Cable Tray Supports in Hall Near Grids E and 2
Member and Nodal Numbers



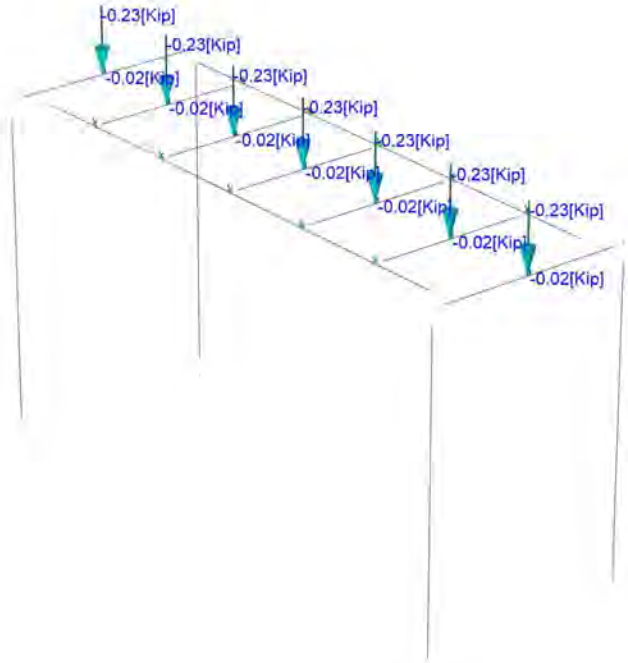
Loads
Distributed user loads - Members

Cable Tray Supports in Hall Near Grids E and 2
Dead Loads



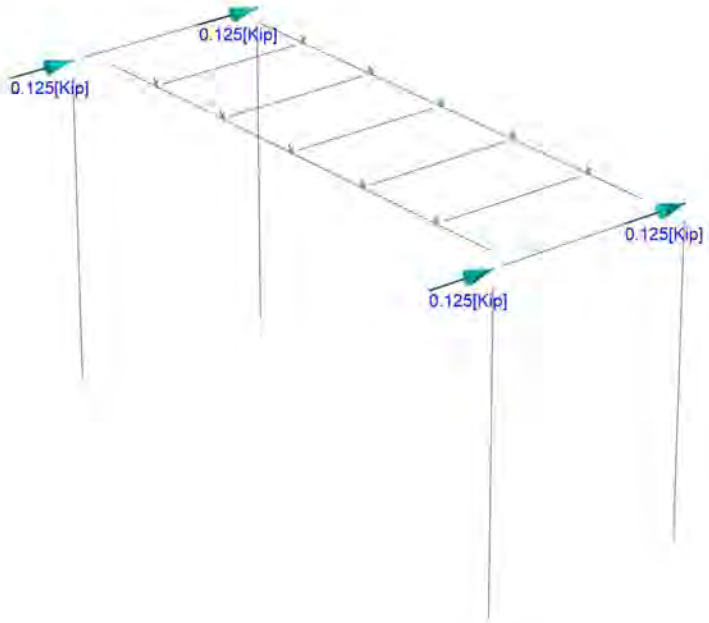
Loads
Concentrated user loads - Members

Cable Tray Supports in Hall Near Grids E and 2
Equipment Loads



Loads
Concentrated user loads - Members

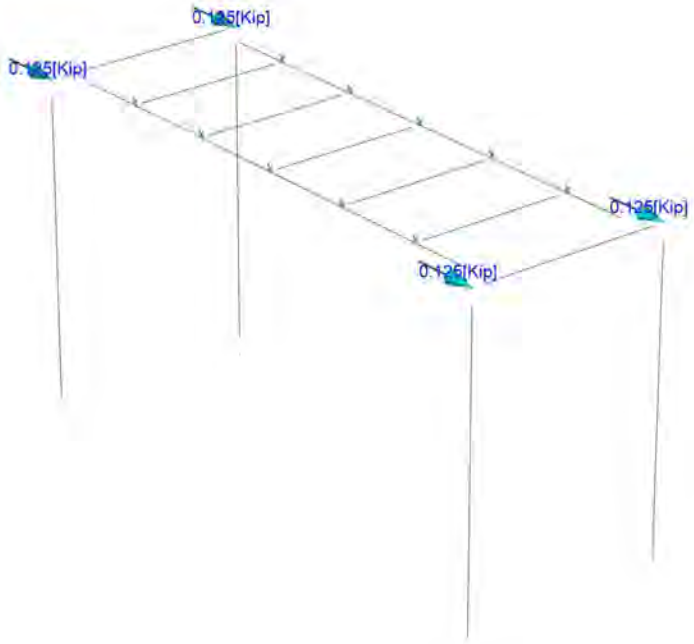
Cable Tray Supports in Hall Near Grids E and 2
Lateral Loads in the X Direction



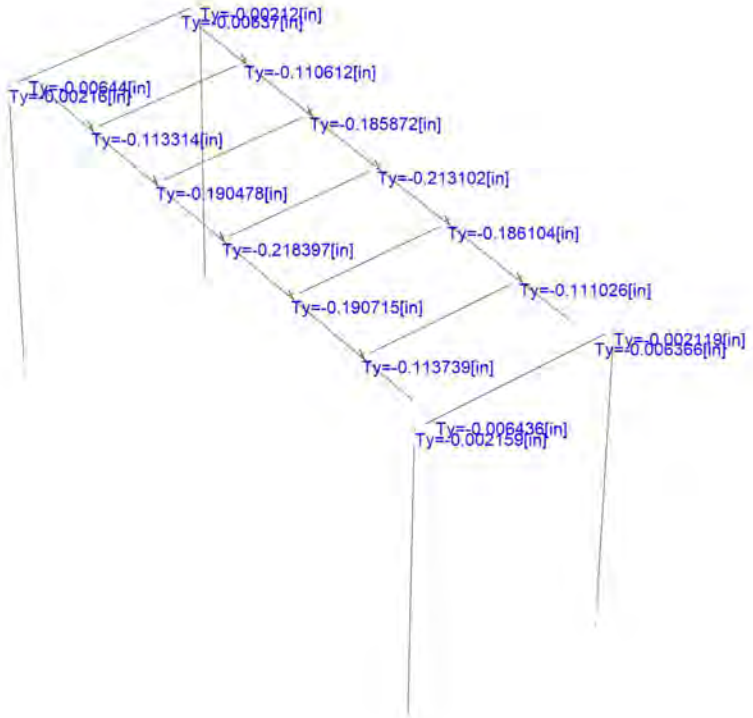
Loads

Concentrated user loads - Members

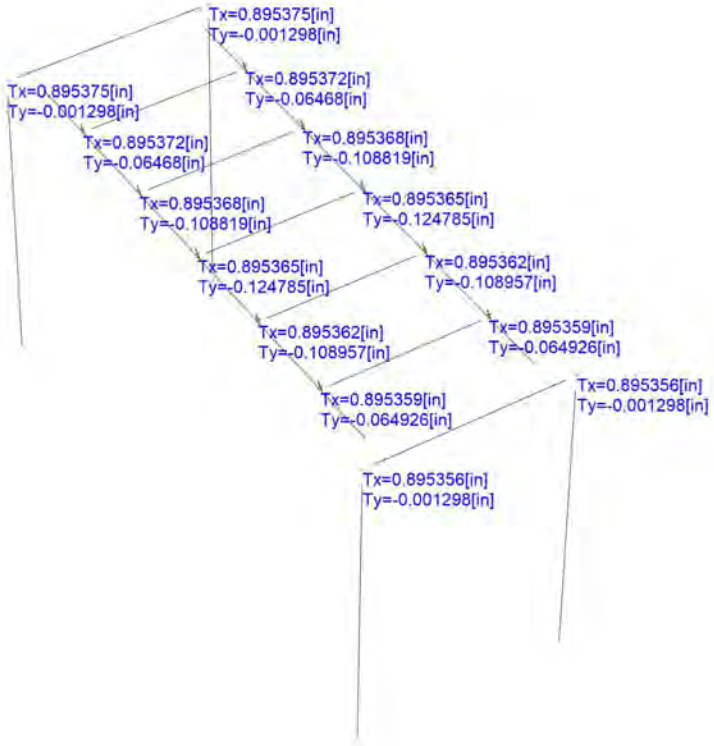
Cable Tray Supports in Hall Near Grids E and 2
Lateral Loads in the Z Direction



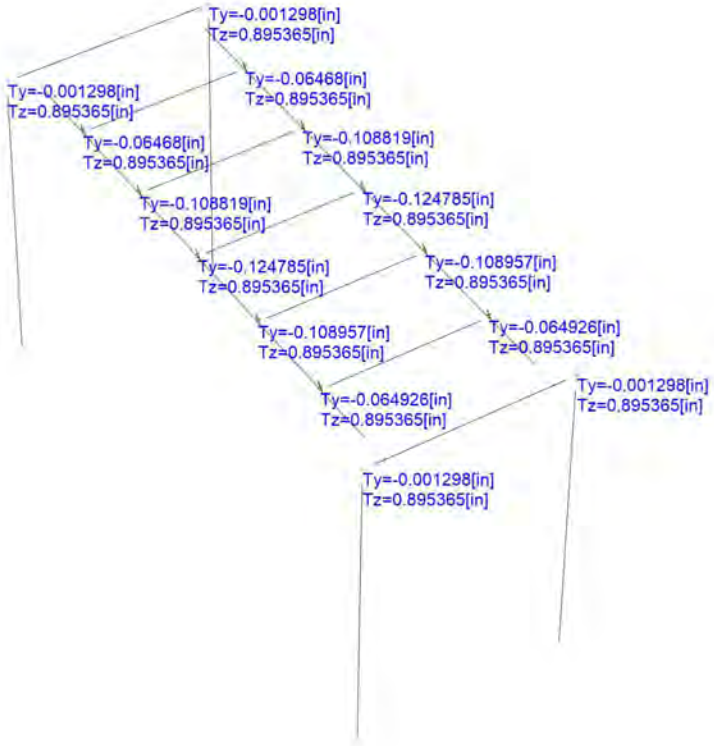
Cable Tray Supports in Hall Near Grids E and 2
Nodal Displacements for DL and LL



Cable Tray Supports in Hall Near Grids E and 2
Nodal Displacements for DL and 0.7E



Cable Tray Supports in Hall Near Grids E and 2
Nodal Displacements for DL and 0.7E



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:48 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Technology Cable Tray Supports in Office Area\Cable Tray Support in Hall Near Grids E and 2.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+E1
D4=1.2DL+E2
D5=1.2DL+E1+LL
D6=1.2DL+E2+LL
D7=0.9DL+E1
D8=0.9DL+E2

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam	***W 10X12***	**9**	D2 at 48.39%	**0.03**	**OK**	
		10	D2 at 48.39%	0.03	OK	
		11	D2 at 48.39%	0.03	OK	
		12	D2 at 48.39%	0.03	OK	
		13	D2 at 48.39%	0.03	OK	
Beam at top of column	***W 12X14***	**5**	D2 at 48.72%	0.08	OK	
		6	D2 at 48.72%	**0.08**	**OK**	
Column	***HSS_SQR 4X4X1_4***	**1**	D6 at 0.00%	0.13	OK	
		2	D5 at 0.00%	0.13	OK	
		3	D6 at 0.00%	**0.13**	**OK**	
		4	D5 at 0.00%	0.13	OK	
Girder	***W 12X19***	**7**	D2 at 49.96%	**0.10**	**OK**	
		8	D2 at 49.96%	0.09	OK	

Current Date: 8/17/2026 1:49 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Technology Cable Tray Supports in Office Area\Cable Tray Support in Hall Near Grids E and 2.retx

Analysis result

Nodes

Nodal displacements envelope

Note.- **Ic** is the controlling load condition
Nodal displacements envelope for :

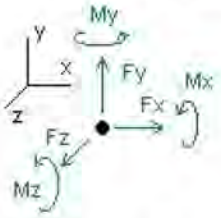
D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+E1
D4=1.2DL+E2
D5=1.2DL+E1+LL
D6=1.2DL+E2+LL
D7=0.9DL+E1
D8=0.9DL+E2
S1=DL
S2=DL+LL
S3=DL+0.75LL
S4=DL+0.7E1
S5=DL+0.7E2
S6=DL+0.7E1
S7=DL+0.7E2
S8=DL+0.75LL+0.525E1
S9=DL+0.75LL+0.525E2
S10=DL+0.525E1
S11=DL+0.525E2
S12=0.6DL+0.7E1
S13=0.6DL+0.7E2

		Translation						Rotation					
Node		**X** [in]	**Ic**	**Y** [in]	**Ic**	**Z** [in]	**Ic**	**Rx** [Rad]	**Ic**	**Ry** [Rad]	**Ic**	**Rz** [Rad]	**Ic**
1	Max	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
	Min	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
3	Max	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
	Min	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
5	Max	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
	Min	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
6	Max	1.329	D5	-0.001	S13	1.330	D6	0.01063	D6	0.00000	D1	0.00000	D1
	Min	0.000	D1	-0.003	D2	0.000	D1	0.00000	D1	0.00000	D1	-0.01063	D5
7	Max	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
	Min	0.000	D1	0.000	D1	0.000	D1	0.00000	D1	0.00000	D1	0.00000	D1
8	Max	1.329	D5	-0.001	S12	1.328	D6	0.01062	D6	0.00000	D1	0.00000	D1
	Min	0.000	D1	-0.003	D2	0.000	D1	0.00000	D1	0.00000	D1	-0.01063	D5
13	Max	1.329	D5	-0.039	S13	1.330	D6	-0.00060	S13	0.00000	D1	-0.00009	S13
	Min	0.000	D1	-0.156	D2	0.000	D1	-0.00241	D2	0.00000	D5	-0.00042	D2

14	Max	1.329	D5	-0.039	S13	1.328	D6	-0.00060	S13	0.00000	D1	0.00042	D2
	Min	0.000	D1	-0.152	D2	0.000	D1	-0.00234	D2	0.00000	D5	0.00009	S13
15	Max	1.329	D5	-0.065	S13	1.330	D6	-0.00034	S13	0.00000	D1	-0.00009	S13
	Min	0.000	D1	-0.262	D2	0.000	D1	-0.00136	D2	0.00000	D5	-0.00042	D2
16	Max	1.329	D5	-0.075	S13	1.330	D6	0.00000	D2	0.00000	D1	-0.00009	S12
	Min	0.000	D1	-0.300	D2	0.000	D1	0.00000	S12	0.00000	D5	-0.00042	D2
17	Max	1.329	D5	-0.065	S13	1.330	D6	0.00137	D2	0.00000	D1	-0.00009	S12
	Min	0.000	D1	-0.261	D2	0.000	D1	0.00034	S13	0.00000	D5	-0.00042	D2
18	Max	1.329	D5	-0.039	S13	1.330	D6	0.00241	D2	0.00000	D1	-0.00009	S12
	Min	0.000	D1	-0.155	D2	0.000	D1	0.00060	S13	0.00000	D5	-0.00042	D2
19	Max	1.329	D5	-0.065	S13	1.328	D6	-0.00034	S12	0.00000	D1	0.00042	D2
	Min	0.000	D1	-0.254	D2	0.000	D1	-0.00132	D2	0.00000	D5	0.00009	S13
20	Max	1.329	D5	-0.075	S13	1.328	D6	0.00000	D2	0.00000	D1	0.00042	D2
	Min	0.000	D1	-0.291	D2	0.000	D1	0.00000	S13	0.00000	D5	0.00009	S12
21	Max	1.329	D5	-0.065	S13	1.328	D6	0.00133	D2	0.00000	D1	0.00042	D2
	Min	0.000	D1	-0.254	D2	0.000	D1	0.00034	S13	0.00000	D5	0.00009	S12
22	Max	1.329	D5	-0.039	S13	1.328	D6	0.00234	D2	0.00000	D1	0.00042	D2
	Min	0.000	D1	-0.151	D2	0.000	D1	0.00060	S13	0.00000	D5	0.00009	S12
2	Max	1.329	D5	-0.001	S12	1.330	D6	0.01063	D6	0.00000	D1	0.00000	D1
	Min	0.000	D1	-0.003	D2	0.000	D1	0.00000	D1	0.00000	D1	-0.01063	D5
4	Max	1.329	D5	-0.001	S13	1.328	D6	0.01062	D6	0.00000	D1	0.00000	D1
	Min	0.000	D1	-0.003	D2	0.000	D1	0.00000	D1	0.00000	D1	-0.01063	D5

Envelope for nodal reactions

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



Direction of positive forces and moments

Envelope of nodal reactions for :

D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+E1
D4=1.2DL+E2
D5=1.2DL+E1+LL
D6=1.2DL+E2+LL
D7=0.9DL+E1
D8=0.9DL+E2
S1=DL
S2=DL+LL

S3=DL+0.75LL
S4=DL+0.7E1
S5=DL+0.7E2
S6=DL+0.7E1
S7=DL+0.7E2
S8=DL+0.75LL+0.525E1
S9=DL+0.75LL+0.525E2
S10=DL+0.525E1
S11=DL+0.525E2
S12=0.6DL+0.7E1
S13=0.6DL+0.7E2

Node		Forces						Moments					
		Fx [Kip]	**lc**	**Fy** [Kip]	**lc**	**Fz** [Kip]	**lc**	**Mx** [Kip*ft]	**lc**	**My** [Kip*ft]	**lc**	**Mz** [Kip*ft]	**lc**
1	Max	0.000	D1	1.633	D2	0.000	D1	0.00000	D1	0.00000	D1	2.09615	D5
	Min	-0.125	D7	0.458	S12	-0.125	D6	-2.09768	D6	0.00000	D1	0.00000	D1
3	Max	0.000	D1	1.600	D2	0.000	D1	0.00000	D1	0.00000	D1	2.09661	D5
	Min	-0.125	D5	0.458	S13	-0.125	D6	-2.09522	D6	0.00000	D1	0.00000	D1
5	Max	0.000	D1	1.634	D2	0.000	D1	0.00000	D1	0.00000	D1	2.09626	D5
	Min	-0.125	D3	0.458	S13	-0.125	D8	-2.09766	D6	0.00000	D1	0.00000	D1
7	Max	0.000	D1	1.601	D2	0.000	D1	0.00000	D1	0.00000	D1	2.09672	D5
	Min	-0.125	D5	0.458	S12	-0.125	D8	-2.09520	D6	0.00000	D1	0.00000	D1

Members

Maximum forces at members

Condition : **D1=1.4DL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.82	0.00	0.00	0.00	0.00	0.00
Min	-1.07	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	-0.82	0.00	0.00	0.00	0.00	0.00
Min	-1.07	0.00	0.00	0.00	0.00	0.00
MEMBER **3**						
Max	-0.82	0.00	0.00	0.00	0.00	0.00
Min	-1.07	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	-0.82	0.00	0.00	0.00	0.00	0.00
Min	-1.07	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	0.00	0.82	0.00	0.00	0.00	1.06
Min	0.00	-0.82	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.82	0.00	0.00	0.00	1.06
Min	0.00	-0.82	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.66	0.00	0.00	0.00	4.84
Min	0.00	-0.66	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.66	0.00	0.00	0.00	4.84
Min	0.00	-0.66	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.12	0.00	0.00	0.00	0.23

Min	0.00	-0.12	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.12	0.00	0.00	0.00	0.23
Min	0.00	-0.12	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.12	0.00	0.00	0.00	0.23
Min	0.00	-0.12	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.12	0.00	0.00	0.00	0.23
Min	0.00	-0.12	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.12	0.00	0.00	0.00	0.23
Min	0.00	-0.12	0.00	0.00	0.00	0.00

Condition : **D2=1.2DL+1.6LL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-1.42	0.00	0.00	0.00	0.00	0.00
Min	-1.63	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	-1.39	0.00	0.00	0.00	0.00	0.00
Min	-1.60	0.00	0.00	0.00	0.00	0.00
MEMBER **3**						
Max	-1.42	0.00	0.00	0.00	0.00	0.00
Min	-1.63	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	-1.39	0.00	0.00	0.00	0.00	0.00
Min	-1.60	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	0.00	1.42	0.00	0.00	0.00	2.38
Min	0.00	-1.39	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	1.42	0.00	0.00	0.00	2.38
Min	0.00	-1.39	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	1.08	0.00	0.00	0.00	8.34
Min	0.00	-1.08	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	1.05	0.00	0.00	0.00	8.09
Min	0.00	-1.05	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.31	0.00	0.00	0.00	0.97
Min	0.00	-0.30	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.31	0.00	0.00	0.00	0.97
Min	0.00	-0.30	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.31	0.00	0.00	0.00	0.97
Min	0.00	-0.30	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.31	0.00	0.00	0.00	0.97
Min	0.00	-0.30	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.31	0.00	0.00	0.00	0.97
Min	0.00	-0.30	0.00	0.00	0.00	0.00

Condition : **D3=1.2DL+E1**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.70	0.12	0.00	0.00	0.00	0.00
Min	-0.92	0.12	0.00	0.00	0.00	-2.05
MEMBER **2**						
Max	-0.70	0.12	0.00	0.00	0.00	0.00
Min	-0.92	0.12	0.00	0.00	0.00	-2.05
MEMBER **3**						
Max	-0.70	0.13	0.00	0.00	0.00	0.00
Min	-0.92	0.13	0.00	0.00	0.00	-2.05
MEMBER **4**						
Max	-0.70	0.13	0.00	0.00	0.00	0.00
Min	-0.92	0.13	0.00	0.00	0.00	-2.05
MEMBER **5**						
Max	0.00	0.70	0.00	0.00	0.00	0.90
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.70	0.00	0.00	0.00	0.91
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.56	0.00	0.00	0.00	4.15
Min	0.00	-0.56	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.56	0.00	0.00	0.00	4.15
Min	0.00	-0.56	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00

Condition : **D4=1.2DL+E2**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.70	0.00	-0.13	0.00	0.00	0.00
Min	-0.92	0.00	-0.13	0.00	-2.05	0.00
MEMBER **2**						
Max	-0.70	0.00	-0.13	0.00	0.00	0.00
Min	-0.92	0.00	-0.13	0.00	-2.05	0.00
MEMBER **3**						
Max	-0.70	0.00	-0.12	0.00	0.00	0.00
Min	-0.92	0.00	-0.12	0.00	-2.05	0.00
MEMBER **4**						
Max	-0.70	0.00	-0.12	0.00	0.00	0.00
Min	-0.92	0.00	-0.12	0.00	-2.05	0.00
MEMBER **5**						
Max	0.00	0.70	0.00	0.00	0.00	0.90
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **6**						

Max	0.00	0.70	0.00	0.00	0.00	0.91
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.56	0.00	0.00	0.00	4.15
Min	0.00	-0.56	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.56	0.00	0.00	0.00	4.15
Min	0.00	-0.56	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.10	0.00	0.00	0.00	0.20
Min	0.00	-0.10	0.00	0.00	0.00	0.00

Condition : **D5=1.2DL+E1+LL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-1.15	0.12	0.00	0.00	0.00	0.00
Min	-1.36	0.12	0.00	0.00	0.00	-2.10
MEMBER **2**						
Max	-1.13	0.13	0.00	0.00	0.00	0.00
Min	-1.34	0.13	0.00	0.00	0.00	-2.10
MEMBER **3**						
Max	-1.15	0.12	0.00	0.00	0.00	0.00
Min	-1.37	0.12	0.00	0.00	0.00	-2.10
MEMBER **4**						
Max	-1.13	0.13	0.00	0.00	0.00	0.00
Min	-1.34	0.13	0.00	0.00	0.00	-2.10
MEMBER **5**						
Max	0.00	1.15	0.00	0.00	0.00	1.82
Min	0.00	-1.13	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	1.15	0.00	0.00	0.00	1.83
Min	0.00	-1.13	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.88	0.00	0.00	0.00	6.77
Min	0.00	-0.88	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.87	0.00	0.00	0.00	6.62
Min	0.00	-0.86	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.23	0.00	0.00	0.00	0.68

Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00

Condition : **D6=1.2DL+E2+LL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-1.15	0.00	-0.13	0.00	0.00	0.00
Min	-1.36	0.00	-0.13	0.00	-2.10	0.00
MEMBER **2**						
Max	-1.13	0.00	-0.13	0.00	0.00	0.00
Min	-1.34	0.00	-0.13	0.00	-2.10	0.00
MEMBER **3**						
Max	-1.15	0.00	-0.12	0.00	0.00	0.00
Min	-1.37	0.00	-0.12	0.00	-2.10	0.00
MEMBER **4**						
Max	-1.13	0.00	-0.12	0.00	0.00	0.00
Min	-1.34	0.00	-0.12	0.00	-2.10	0.00
MEMBER **5**						
Max	0.00	1.15	0.00	0.00	0.00	1.82
Min	0.00	-1.13	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	1.15	0.00	0.00	0.00	1.83
Min	0.00	-1.13	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.88	0.00	0.00	0.00	6.77
Min	0.00	-0.88	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.87	0.00	0.00	0.00	6.62
Min	0.00	-0.86	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.23	0.00	0.00	0.00	0.68
Min	0.00	-0.22	0.00	0.00	0.00	0.00

Condition : **D7=0.9DL+E1**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.53	0.12	0.00	0.00	0.00	0.00
Min	-0.69	0.12	0.00	0.00	0.00	-2.02
MEMBER **2**						
Max	-0.53	0.12	0.00	0.00	0.00	0.00
Min	-0.69	0.12	0.00	0.00	0.00	-2.02
MEMBER **3**						
Max	-0.53	0.12	0.00	0.00	0.00	0.00
Min	-0.69	0.12	0.00	0.00	0.00	-2.02
MEMBER **4**						
Max	-0.53	0.12	0.00	0.00	0.00	0.00
Min	-0.69	0.12	0.00	0.00	0.00	-2.02
MEMBER **5**						
Max	0.00	0.53	0.00	0.00	0.00	0.68
Min	0.00	-0.53	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.53	0.00	0.00	0.00	0.68
Min	0.00	-0.53	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.42	0.00	0.00	0.00	3.11
Min	0.00	-0.42	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.42	0.00	0.00	0.00	3.11
Min	0.00	-0.42	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00

Condition : **D8=0.9DL+E2**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.53	0.00	-0.13	0.00	0.00	0.00
Min	-0.69	0.00	-0.13	0.00	-2.02	0.00
MEMBER **2**						
Max	-0.53	0.00	-0.13	0.00	0.00	0.00
Min	-0.69	0.00	-0.13	0.00	-2.02	0.00
MEMBER **3**						
Max	-0.53	0.00	-0.12	0.00	0.00	0.00
Min	-0.69	0.00	-0.12	0.00	-2.02	0.00
MEMBER **4**						
Max	-0.53	0.00	-0.12	0.00	0.00	0.00
Min	-0.69	0.00	-0.12	0.00	-2.02	0.00
MEMBER **5**						
Max	0.00	0.53	0.00	0.00	0.00	0.68
Min	0.00	-0.53	0.00	0.00	0.00	0.00
MEMBER **6**						

Max	0.00	0.53	0.00	0.00	0.00	0.68
Min	0.00	-0.53	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.42	0.00	0.00	0.00	3.11
Min	0.00	-0.42	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.42	0.00	0.00	0.00	3.11
Min	0.00	-0.42	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.08	0.00	0.00	0.00	0.15
Min	0.00	-0.08	0.00	0.00	0.00	0.00

Condition : **S1=DL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.59	0.00	0.00	0.00	0.00	0.00
Min	-0.76	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	-0.59	0.00	0.00	0.00	0.00	0.00
Min	-0.76	0.00	0.00	0.00	0.00	0.00
MEMBER **3**						
Max	-0.59	0.00	0.00	0.00	0.00	0.00
Min	-0.76	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	-0.59	0.00	0.00	0.00	0.00	0.00
Min	-0.76	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16

Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S2=DL+LL**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-1.03	0.00	0.00	0.00	0.00	0.00
Min	-1.21	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	-1.01	0.00	0.00	0.00	0.00	0.00
Min	-1.19	0.00	0.00	0.00	0.00	0.00
MEMBER **3**						
Max	-1.03	0.00	0.00	0.00	0.00	0.00
Min	-1.21	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	-1.01	0.00	0.00	0.00	0.00	0.00
Min	-1.19	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	0.00	1.03	0.00	0.00	0.00	1.67
Min	0.00	-1.01	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	1.03	0.00	0.00	0.00	1.67
Min	0.00	-1.01	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.79	0.00	0.00	0.00	6.07
Min	0.00	-0.79	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.77	0.00	0.00	0.00	5.92
Min	0.00	-0.77	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.21	0.00	0.00	0.00	0.65
Min	0.00	-0.21	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.21	0.00	0.00	0.00	0.65
Min	0.00	-0.21	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.21	0.00	0.00	0.00	0.65
Min	0.00	-0.21	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.21	0.00	0.00	0.00	0.65
Min	0.00	-0.21	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.21	0.00	0.00	0.00	0.65
Min	0.00	-0.21	0.00	0.00	0.00	0.00

Condition : **S3=DL+0.75LL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.92	0.00	0.00	0.00	0.00	0.00
Min	-1.10	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	-0.91	0.00	0.00	0.00	0.00	0.00
Min	-1.08	0.00	0.00	0.00	0.00	0.00
MEMBER **3**						
Max	-0.92	0.00	0.00	0.00	0.00	0.00
Min	-1.10	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	-0.91	0.00	0.00	0.00	0.00	0.00
Min	-1.08	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.71	0.00	0.00	0.00	5.42
Min	0.00	-0.71	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.70	0.00	0.00	0.00	5.31
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00

Condition : **S4=DL+0.7E1**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **2**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **3**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **4**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						

Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S5=DL+0.7E2**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **2**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **3**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **4**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16

Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S6=DL+0.7E1**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **2**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **3**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **4**						
Max	-0.59	0.09	0.00	0.00	0.00	0.00
Min	-0.76	0.09	0.00	0.00	0.00	-1.42
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S7=DL+0.7E2**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **2**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **3**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **4**						
Max	-0.59	0.00	-0.09	0.00	0.00	0.00
Min	-0.76	0.00	-0.09	0.00	-1.42	0.00
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S8=DL+0.75LL+0.525E1**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.92	0.07	0.00	0.00	0.00	0.00
Min	-1.10	0.07	0.00	0.00	0.00	-1.09
MEMBER **2**						
Max	-0.91	0.07	0.00	0.00	0.00	0.00
Min	-1.08	0.07	0.00	0.00	0.00	-1.09
MEMBER **3**						
Max	-0.92	0.07	0.00	0.00	0.00	0.00
Min	-1.10	0.07	0.00	0.00	0.00	-1.09
MEMBER **4**						
Max	-0.91	0.07	0.00	0.00	0.00	0.00
Min	-1.08	0.07	0.00	0.00	0.00	-1.09
MEMBER **5**						
Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **6**						

Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.71	0.00	0.00	0.00	5.42
Min	0.00	-0.71	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.70	0.00	0.00	0.00	5.31
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00

Condition : **S9=DL+0.75LL+0.525E2**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.92	0.00	-0.07	0.00	0.00	0.00
Min	-1.10	0.00	-0.07	0.00	-1.09	0.00
MEMBER **2**						
Max	-0.91	0.00	-0.07	0.00	0.00	0.00
Min	-1.08	0.00	-0.07	0.00	-1.09	0.00
MEMBER **3**						
Max	-0.92	0.00	-0.07	0.00	0.00	0.00
Min	-1.10	0.00	-0.07	0.00	-1.09	0.00
MEMBER **4**						
Max	-0.91	0.00	-0.07	0.00	0.00	0.00
Min	-1.08	0.00	-0.07	0.00	-1.09	0.00
MEMBER **5**						
Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.92	0.00	0.00	0.00	1.44
Min	0.00	-0.91	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.71	0.00	0.00	0.00	5.42
Min	0.00	-0.71	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.70	0.00	0.00	0.00	5.31
Min	0.00	-0.70	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.18	0.00	0.00	0.00	0.53

Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.18	0.00	0.00	0.00	0.53
Min	0.00	-0.18	0.00	0.00	0.00	0.00

Condition : **S10=DL+0.525E1**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.59	0.07	0.00	0.00	0.00	0.00
Min	-0.76	0.07	0.00	0.00	0.00	-1.07
MEMBER **2**						
Max	-0.59	0.07	0.00	0.00	0.00	0.00
Min	-0.76	0.07	0.00	0.00	0.00	-1.07
MEMBER **3**						
Max	-0.59	0.07	0.00	0.00	0.00	0.00
Min	-0.76	0.07	0.00	0.00	0.00	-1.07
MEMBER **4**						
Max	-0.59	0.07	0.00	0.00	0.00	0.00
Min	-0.76	0.07	0.00	0.00	0.00	-1.07
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S11=DL+0.525E2**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.59	0.00	-0.07	0.00	0.00	0.00
Min	-0.76	0.00	-0.07	0.00	-1.07	0.00
MEMBER **2**						
Max	-0.59	0.00	-0.07	0.00	0.00	0.00
Min	-0.76	0.00	-0.07	0.00	-1.07	0.00
MEMBER **3**						
Max	-0.59	0.00	-0.07	0.00	0.00	0.00
Min	-0.76	0.00	-0.07	0.00	-1.07	0.00
MEMBER **4**						
Max	-0.59	0.00	-0.07	0.00	0.00	0.00
Min	-0.76	0.00	-0.07	0.00	-1.07	0.00
MEMBER **5**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.59	0.00	0.00	0.00	0.75
Min	0.00	-0.59	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.47	0.00	0.00	0.00	3.46
Min	0.00	-0.47	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.09	0.00	0.00	0.00	0.16
Min	0.00	-0.09	0.00	0.00	0.00	0.00

Condition : **S12=0.6DL+0.7E1**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-0.35	0.09	0.00	0.00	0.00	0.00
Min	-0.46	0.09	0.00	0.00	0.00	-1.40
MEMBER **2**						
Max	-0.35	0.09	0.00	0.00	0.00	0.00
Min	-0.46	0.09	0.00	0.00	0.00	-1.40
MEMBER **3**						
Max	-0.35	0.09	0.00	0.00	0.00	0.00
Min	-0.46	0.09	0.00	0.00	0.00	-1.40
MEMBER **4**						
Max	-0.35	0.09	0.00	0.00	0.00	0.00
Min	-0.46	0.09	0.00	0.00	0.00	-1.40
MEMBER **5**						
Max	0.00	0.35	0.00	0.00	0.00	0.45
Min	0.00	-0.35	0.00	0.00	0.00	0.00
MEMBER **6**						

Max	0.00	0.35	0.00	0.00	0.00	0.45
Min	0.00	-0.35	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.28	0.00	0.00	0.00	2.08
Min	0.00	-0.28	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.28	0.00	0.00	0.00	2.08
Min	0.00	-0.28	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00

Condition : **S13=0.6DL+0.7E2**

	Axial [Kip]	**Shear V2** [Kip]	**Shear V3** [Kip]	**Torsion** [Kip*ft]	**M22** [Kip*ft]	**M33** [Kip*ft]
MEMBER **1**						
Max	-0.35	0.00	-0.09	0.00	0.00	0.00
Min	-0.46	0.00	-0.09	0.00	-1.40	0.00
MEMBER **2**						
Max	-0.35	0.00	-0.09	0.00	0.00	0.00
Min	-0.46	0.00	-0.09	0.00	-1.40	0.00
MEMBER **3**						
Max	-0.35	0.00	-0.09	0.00	0.00	0.00
Min	-0.46	0.00	-0.09	0.00	-1.40	0.00
MEMBER **4**						
Max	-0.35	0.00	-0.09	0.00	0.00	0.00
Min	-0.46	0.00	-0.09	0.00	-1.40	0.00
MEMBER **5**						
Max	0.00	0.35	0.00	0.00	0.00	0.45
Min	0.00	-0.35	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	0.00	0.35	0.00	0.00	0.00	0.45
Min	0.00	-0.35	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	0.00	0.28	0.00	0.00	0.00	2.08
Min	0.00	-0.28	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	0.00	0.28	0.00	0.00	0.00	2.08
Min	0.00	-0.28	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	0.00	0.05	0.00	0.00	0.00	0.10

Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	0.00	0.05	0.00	0.00	0.00	0.10
Min	0.00	-0.05	0.00	0.00	0.00	0.00

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **D1=1.4DL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.00000	(< L/10000)	0.00000	0.00000	(< L/10000)	0.00000
2	0.00000	(< L/10000)	0.00000	0.00000	(< L/10000)	0.00000
3	0.00000	(< L/10000)	0.00000	0.00000	(< L/10000)	0.00000
4	0.00000	(< L/10000)	0.00000	0.00000	(< L/10000)	0.00000
5	0.00831	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
6	0.00831	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
7	0.16981	(L/1915)	50.00000	0.00000	(< L/10000)	0.00000
8	0.16981	(L/1915)	50.00000	0.00000	(< L/10000)	0.00000
9	0.00173	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
10	0.00173	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
11	0.00173	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
12	0.00173	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
13	0.00173	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D2=1.2DL+1.6LL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
2	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
3	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
4	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
5	0.01675	(L/6986)	50.00000	0.00000	(< L/10000)	0.00000
6	0.01676	(L/6982)	50.00000	0.00000	(< L/10000)	0.00000
7	0.29062	(L/1119)	50.00000	0.00000	(< L/10000)	0.00000
8	0.28226	(L/1152)	50.00000	0.00000	(< L/10000)	0.00000
9	0.00621	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
10	0.00621	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
11	0.00621	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
12	0.00621	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
13	0.00621	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D3=1.2DL+E1**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.24587	(L/765)	37.50000	0.00000	(< L/10000)	0.00000
2	0.24587	(L/765)	37.50000	0.00000	(< L/10000)	0.00000
3	0.24588	(L/765)	37.50000	0.00000	(< L/10000)	0.00000
4	0.24588	(L/765)	37.50000	0.00000	(< L/10000)	0.00000
5	0.00712	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000
6	0.00712	(< L/10000)	50.00000	0.00000	(< L/10000)	0.00000

7	0.14555	(L/2235)	50.00000	0.00000	(< L/10000)	50.00000
8	0.14555	(L/2235)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **D4=1.2DL+E2**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.00000	(< L/10000)	50.00000	0.24588	(L/765)	37.50000
2	0.00000	(< L/10000)	50.00000	0.24588	(L/765)	37.50000
3	0.00000	(< L/10000)	50.00000	0.24588	(L/765)	37.50000
4	0.00000	(< L/10000)	50.00000	0.24588	(L/765)	37.50000
5	0.00712	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
6	0.00712	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
7	0.14555	(L/2235)	50.00000	0.00000	(< L/10000)	50.00000
8	0.14555	(L/2235)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
10	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
11	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
12	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
13	0.00149	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000

CONDITION **D5=1.2DL+E1+LL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.25338	(L/742)	37.50000	0.00000	(< L/10000)	25.00000
2	0.25337	(L/742)	37.50000	0.00000	(< L/10000)	25.00000
3	0.25339	(L/742)	37.50000	0.00000	(< L/10000)	25.00000
4	0.25339	(L/742)	37.50000	0.00000	(< L/10000)	25.00000
5	0.01314	(L/8906)	50.00000	0.00000	(< L/10000)	25.00000
6	0.01314	(L/8901)	50.00000	0.00000	(< L/10000)	25.00000
7	0.23622	(L/1377)	50.00000	0.00000	(< L/10000)	25.00000
8	0.23099	(L/1408)	50.00000	0.00000	(< L/10000)	25.00000
9	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
10	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
11	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
12	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
13	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000

CONDITION **D6=1.2DL+E2+LL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.00000	(< L/10000)	50.00000	0.25356	(L/741)	37.50000
2	0.00000	(< L/10000)	50.00000	0.25320	(L/742)	37.50000
3	0.00000	(< L/10000)	50.00000	0.25356	(L/741)	37.50000
4	0.00000	(< L/10000)	50.00000	0.25320	(L/742)	37.50000
5	0.01314	(L/8906)	50.00000	0.00000	(< L/10000)	50.00000
6	0.01314	(L/8901)	50.00000	0.00000	(< L/10000)	50.00000
7	0.23622	(L/1377)	50.00000	0.00000	(< L/10000)	50.00000
8	0.23099	(L/1408)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

11	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00444	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **D7=0.9DL+E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.24255	(L/775)	37.50000	0.00000	(< L/10000)	50.00000
2	0.24255	(L/775)	37.50000	0.00000	(< L/10000)	50.00000
3	0.24255	(L/775)	37.50000	0.00000	(< L/10000)	50.00000
4	0.24255	(L/775)	37.50000	0.00000	(< L/10000)	50.00000
5	0.00534	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
6	0.00534	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
7	0.10916	(L/2980)	50.00000	0.00000	(< L/10000)	50.00000
8	0.10916	(L/2980)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **D8=0.9DL+E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.24255	(L/775)	37.50000
2	0.00000	(< L/10000)	50.00000	0.24255	(L/775)	37.50000
3	0.00000	(< L/10000)	50.00000	0.24255	(L/775)	37.50000
4	0.00000	(< L/10000)	50.00000	0.24255	(L/775)	37.50000
5	0.00534	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
6	0.00534	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.10916	(L/2980)	50.00000	0.00000	(< L/10000)	75.00000
8	0.10916	(L/2980)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00111	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S1=DL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
2	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
3	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
4	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S2=DL+LL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
2	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
3	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
4	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
5	0.01195	(L/9790)	50.00000	0.00000	(< L/10000)	75.00000
6	0.01196	(L/9785)	50.00000	0.00000	(< L/10000)	75.00000
7	0.21196	(L/1535)	50.00000	0.00000	(< L/10000)	75.00000
8	0.20673	(L/1573)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00419	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00419	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00419	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00419	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00419	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S3=DL+0.75LL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
2	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
3	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
4	0.00000	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
5	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.18929	(L/1718)	50.00000	0.00000	(< L/10000)	75.00000
8	0.18537	(L/1755)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S4=DL+0.7E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.17055	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
2	0.17055	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
3	0.17056	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
4	0.17056	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **S5=DL+0.7E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
2	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
3	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
4	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S6=DL+0.7E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.17055	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
2	0.17055	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
3	0.17056	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
4	0.17056	(L/1102)	37.50000	0.00000	(< L/10000)	75.00000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **S7=DL+0.7E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
2	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
3	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
4	0.00000	(< L/10000)	50.00000	0.17056	(L/1102)	37.50000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

CONDITION **S8=DL+0.75LL+0.525E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.13080	(L/1437)	37.50000	0.00000	(< L/10000)	75.00000
2	0.13079	(L/1437)	37.50000	0.00000	(< L/10000)	75.00000
3	0.13080	(L/1437)	37.50000	0.00000	(< L/10000)	75.00000
4	0.13080	(L/1437)	37.50000	0.00000	(< L/10000)	75.00000
5	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.18929	(L/1718)	50.00000	0.00000	(< L/10000)	25.00000
8	0.18537	(L/1755)	50.00000	0.00000	(< L/10000)	25.00000
9	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
10	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
11	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
12	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
13	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000

CONDITION **S9=DL+0.75LL+0.525E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.13087	(L/1437)	37.50000
2	0.00000	(< L/10000)	50.00000	0.13073	(L/1438)	37.50000
3	0.00000	(< L/10000)	50.00000	0.13087	(L/1437)	37.50000
4	0.00000	(< L/10000)	50.00000	0.13073	(L/1438)	37.50000
5	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
6	0.01045	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
7	0.18929	(L/1718)	50.00000	0.00000	(< L/10000)	50.00000
8	0.18537	(L/1755)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00345	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **S10=DL+0.525E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.12792	(L/1470)	37.50000	0.00000	(< L/10000)	50.00000
2	0.12792	(L/1470)	37.50000	0.00000	(< L/10000)	50.00000
3	0.12792	(L/1470)	37.50000	0.00000	(< L/10000)	50.00000
4	0.12792	(L/1470)	37.50000	0.00000	(< L/10000)	50.00000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **S11=DL+0.525E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.12792	(L/1470)	37.50000
2	0.00000	(< L/10000)	50.00000	0.12792	(L/1470)	37.50000
3	0.00000	(< L/10000)	50.00000	0.12792	(L/1470)	37.50000
4	0.00000	(< L/10000)	50.00000	0.12792	(L/1470)	37.50000
5	0.00593	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
6	0.00594	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
7	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	25.00000
8	0.12129	(L/2682)	50.00000	0.00000	(< L/10000)	25.00000
9	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
10	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
11	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
12	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
13	0.00124	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000

CONDITION **S12=0.6DL+0.7E1**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.16752	(L/1122)	37.50000	0.00000	(< L/10000)	25.00000
2	0.16752	(L/1122)	37.50000	0.00000	(< L/10000)	25.00000
3	0.16752	(L/1122)	37.50000	0.00000	(< L/10000)	25.00000
4	0.16752	(L/1122)	37.50000	0.00000	(< L/10000)	25.00000
5	0.00356	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
6	0.00356	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
7	0.07277	(L/4469)	50.00000	0.00000	(< L/10000)	50.00000
8	0.07277	(L/4469)	50.00000	0.00000	(< L/10000)	50.00000
9	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
10	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
11	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
12	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000
13	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	50.00000

CONDITION **S13=0.6DL+0.7E2**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.00000	(< L/10000)	50.00000	0.16752	(L/1122)	37.50000
2	0.00000	(< L/10000)	50.00000	0.16752	(L/1122)	37.50000
3	0.00000	(< L/10000)	50.00000	0.16752	(L/1122)	37.50000
4	0.00000	(< L/10000)	50.00000	0.16752	(L/1122)	37.50000
5	0.00356	(< L/10000)	50.00000	0.00000	(< L/10000)	25.00000
6	0.00356	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
7	0.07277	(L/4469)	50.00000	0.00000	(< L/10000)	75.00000
8	0.07277	(L/4469)	50.00000	0.00000	(< L/10000)	75.00000
9	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
10	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
11	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
12	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000
13	0.00074	(< L/10000)	50.00000	0.00000	(< L/10000)	75.00000

RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:50 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Technology Cable Tray Supports in Office Area\Cable Tray Support in Hall Near Grids E and 2.retx

Steel Code Check
Concise Report

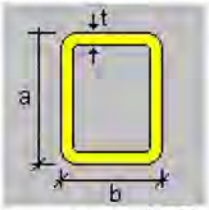
AISC 360-2016 LRFD (Hot-rolled)

Member : 3 (Column) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	15.67

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
15.67	15.67

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
15.67	15.67	15.67	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.03			
Capacity	:	49.86 [Kip]	Reference	:	Cl.E3
Demand	:	1.63 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	49.86	Cl.E3

Compression in the minor axis 22

Ratio	:	0.03			
Capacity	:	49.86 [Kip]	Reference	:	Cl.E3
Demand	:	1.63 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	49.86	Cl.E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.12			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-2.10 [Kip*ft]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.4

Bending about minor axis, M22

Ratio	:	0.12			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	2.10 [Kip*ft]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.12 [Kip]	Ctrl Eq.	:	D8 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.13 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.13			
Ctrl Eq.	:	D6 at 0.00%	Reference	:	Eq.H1-1b

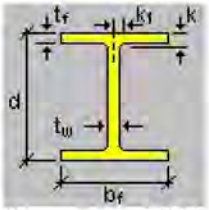
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.13	Eq.H1-1b

Member : 6 (Beam at top of column) - OK

Section information

Section name: W 12X14 **(US)**

Dimensions



bf	=	3.970	[in]	Width
d	=	11.900	[in]	Depth
k	=	0.525	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.225	[in]	Flange thickness
tw	=	0.200	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.160	
Moment of Inertia (local axes) (I)	[in4]	88.600	2.360
Moment of Inertia (principal axes) (I')	[in4]	88.600	2.360
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.615	0.753
Radius of gyration (principal axes) (r')	[in]	4.615	0.753
Saint-Venant torsion constant. (J)	[in4]	0.070	
Section warping constant. (Cw)	[in6]	80.400	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.900	1.190
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.900	1.190
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.900	1.190
Plastic section modulus (local axis) (Z)	[in3]	17.400	1.900
Plastic section modulus (principal axis) (Z')	[in3]	17.400	1.900
Polar radius of gyration. (ro)	[in]	4.676	
Area for shear (Aw)	[in2]	1.790	2.380
Torsional constant. (C)	[in3]	0.272	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	9.75

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
9.75	9.75

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
9.75	9.75	9.75	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 187.20 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	187.20	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 154.80 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D5 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	154.80	Cl.E3

Compression in the minor axis 22

Ratio	: 0.00			
Capacity	: 38.95 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D5 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	38.95	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	85.66	Cl.E4

Flexural Design ✔

Bending about major axis, M33

Ratio	: 0.08			
Capacity	: 29.67 [Kip*ft]	Reference	: Cl.F2.2	
Demand	: 2.38 [Kip*ft]	Ctrl Eq.	: D2 at 48.72%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	65.25	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	29.67	Cl.F2.2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	7.13 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D6 at 10.26%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	7.13	Cl.F6.1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	48.24 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	48.24	Cl.G1

Shear in minor axis 22

Ratio	:	0.02			
Capacity	:	64.26 [Kip]	Reference	:	Cl.G1
Demand	:	1.42 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	64.26	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.08			
Ctrl Eq.	:	D2 at 48.72%	Reference	:	Eq.H1-1b

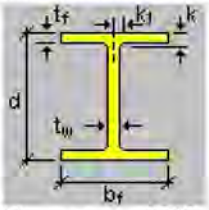
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.08	Eq.H1-1b

Member : 7 (Girder) - OK

Section information

Section name: W 12X19 **(US)**

Dimensions



bf	=	4.010	[in]	Width
d	=	12.200	[in]	Depth
k	=	0.650	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.350	[in]	Flange thickness
tw	=	0.235	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	5.570	
Moment of Inertia (local axes) (I)	[in4]	130.000	3.760
Moment of Inertia (principal axes) (I')	[in4]	130.000	3.760
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.831	0.822
Radius of gyration (principal axes) (r')	[in]	4.831	0.822
Saint-Venant torsion constant. (J)	[in4]	0.180	
Section warping constant. (Cw)	[in6]	131.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	21.300	1.880
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	21.300	1.880
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	21.300	1.880
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	21.300	1.880
Plastic section modulus (local axis) (Z)	[in3]	24.700	2.980
Plastic section modulus (principal axis) (Z')	[in3]	24.700	2.980
Polar radius of gyration. (ro)	[in]	4.900	
Area for shear (Aw)	[in2]	2.810	2.870
Torsional constant. (C)	[in3]	0.474	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	27.10

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
4.52	27.10
4.52	
4.52	
4.52	
4.52	
4.50	

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
27.10	4.52	27.10	1.0	1.0	1.0
	4.52			1.0	
	4.52			1.0	
	4.52			1.0	
	4.52			1.0	
	4.50			1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	250.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	250.65	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	174.77 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 49.96%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	174.77	Cl.E3

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	77.64 [Kip]	Reference	:	Cl.E4
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 49.96%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	176.64	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	77.64	Cl.E4

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.10			
Capacity	:	85.61 [Kip*ft]	Reference	:	Cl.F2.2
Demand	:	8.34 [Kip*ft]	Ctrl Eq.	:	D2 at 49.96%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	92.63	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	85.61	Cl.F2.2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	11.18 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D5 at 16.60%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	11.18	Cl.F6.1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	75.79 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D5 at 16.60%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	75.79	Cl.G1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	86.10 [Kip]	Reference	:	Cl.G1
Demand	:	1.08 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	86.10	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.10			
Ctrl Eq.	:	D2 at 49.96%	Reference	:	Eq.H1-1b

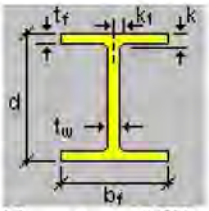
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	–	0.10	Eq.H1-1b

Member : 9 (Beam) - OK

Section information

Section name: W 10X12 **(US)**

Dimensions



bf	=	3.960	[in]	Width
d	=	9.870	[in]	Depth
k	=	0.510	[in]	Distance k
k1	=	0.563	[in]	Distance k1
tf	=	0.210	[in]	Flange thickness
tw	=	0.190	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.540	
Moment of Inertia (local axes) (I)	[in4]	53.800	2.180
Moment of Inertia (principal axes) (I')	[in4]	53.800	2.180
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.898	0.785
Radius of gyration (principal axes) (r')	[in]	3.898	0.785
Saint-Venant torsion constant. (J)	[in4]	0.055	
Section warping constant. (Cw)	[in6]	50.900	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	10.900	1.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	10.900	1.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	10.900	1.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	10.900	1.100
Plastic section modulus (local axis) (Z)	[in3]	12.600	1.740
Plastic section modulus (principal axis) (Z')	[in3]	12.600	1.740
Polar radius of gyration. (ro)	[in]	3.977	
Area for shear (Aw)	[in2]	1.660	1.880
Torsional constant. (C)	[in3]	0.222	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	7.75

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
7.75	7.75

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
7.75	7.75	7.75	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	159.30 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	159.30	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	141.07 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	141.07	Cl.E3

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	56.94 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	56.94	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	95.43	Cl.E4

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.03			
Capacity	:	38.12 [Kip*ft]	Reference	:	Cl.F2.2
Demand	:	0.97 [Kip*ft]	Ctrl Eq.	:	D2 at 48.39%

Intermediate results	Unit	Value	Reference
Section classification			
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	38.12	Cl.F2.2
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	46.90	Cl.F3.1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	6.46 [Kip*ft]	Reference	:	Cl.F6.2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	6.53	Cl.F6.1
Factored compression flange local buckling strength about a geometric ...	[Kip*ft]	6.46	Cl.F6.2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	44.91 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	44.91	Cl.G1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	56.40 [Kip]	Reference	:	Cl.G1
Demand	:	0.31 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	56.40	Cl.G1

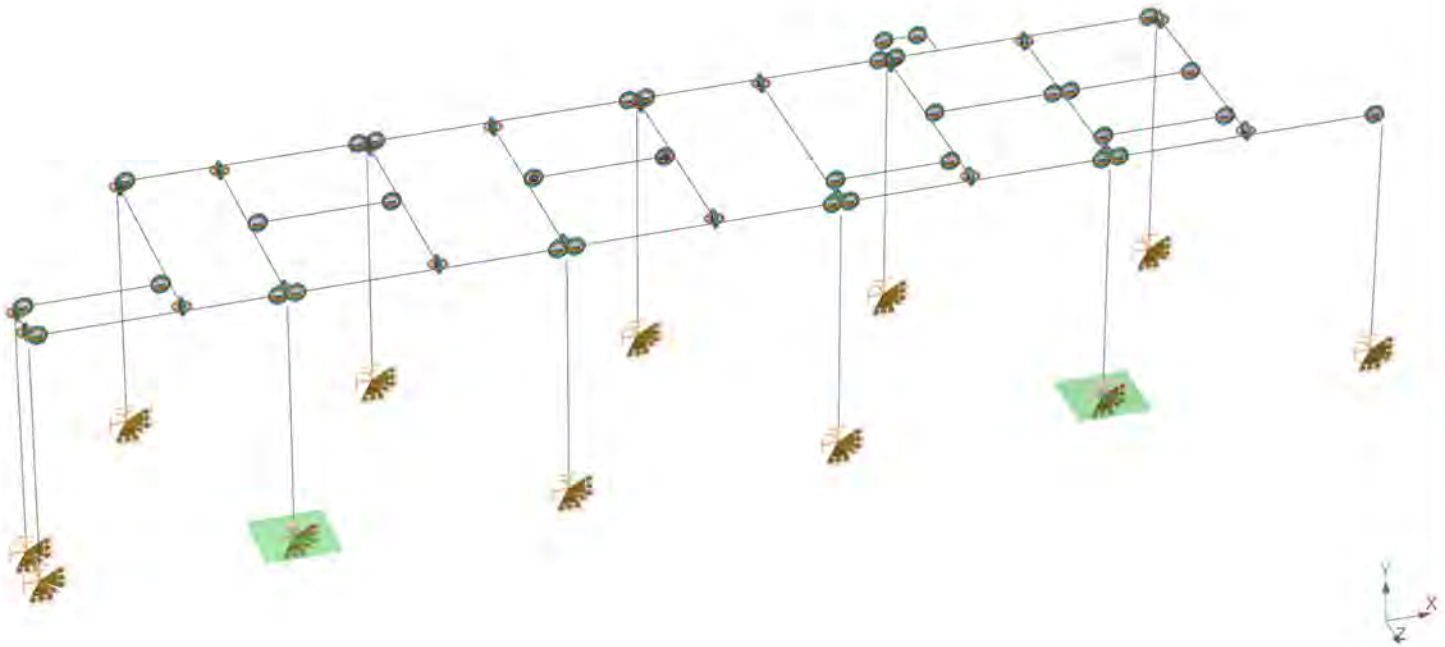
Combined Actions Design ✓

Combined flexure and axial

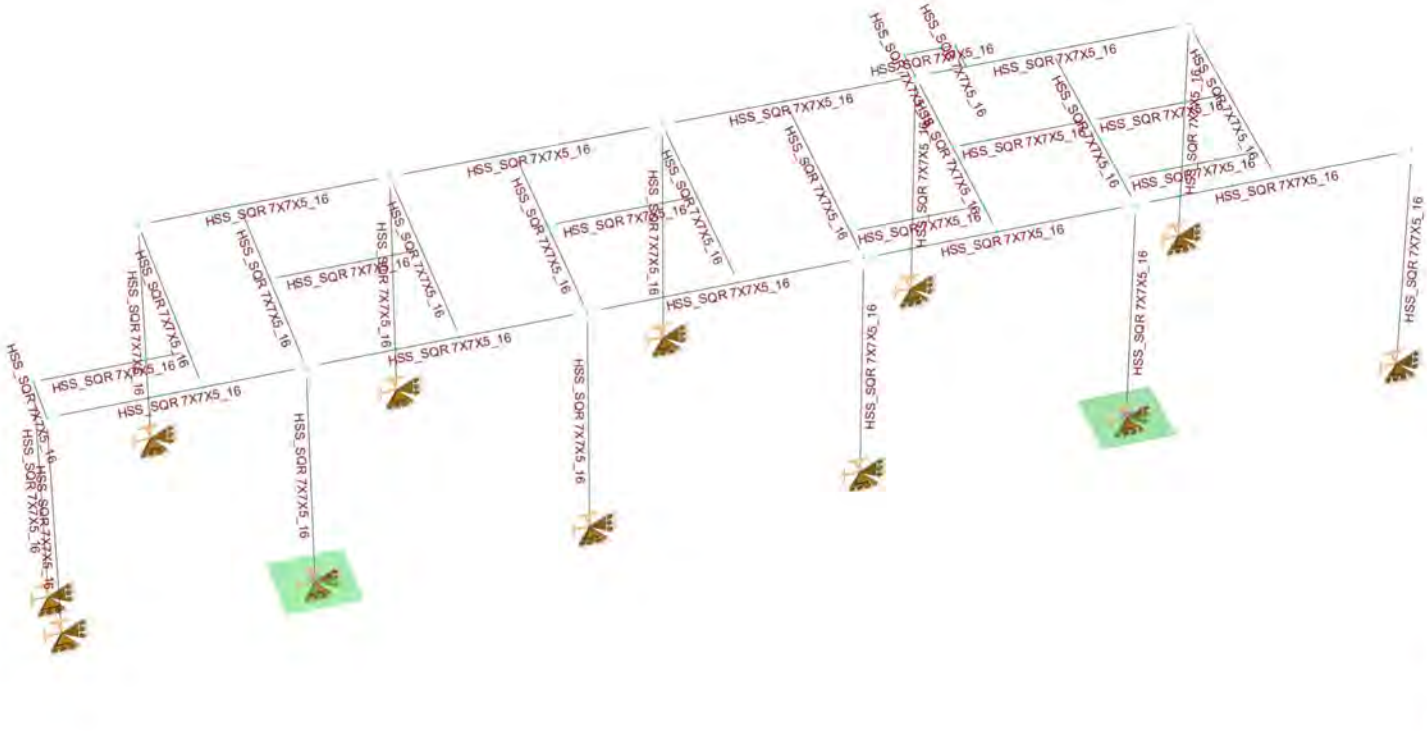
Ratio	:	0.03			
Ctrl Eq.	:	D2 at 48.39%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	---	0.03	Eq.H1-1b

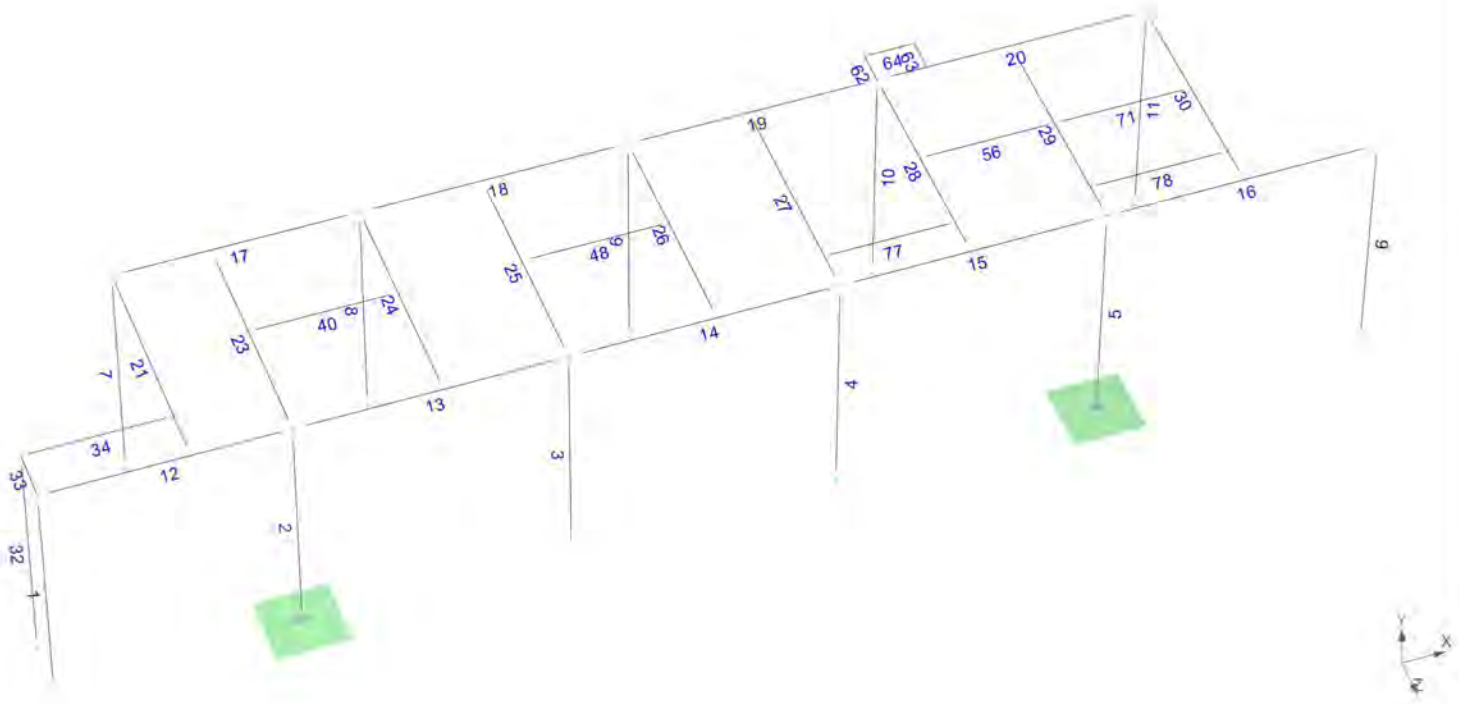
Exterior Cable Tray Supports in Loading Dock
Near Grids 13 and 15
Column Supports and Beam Hinges



Exterior Cable Tray Supports in Loading Dock
Near Grids 13 and 15
Member Sizes

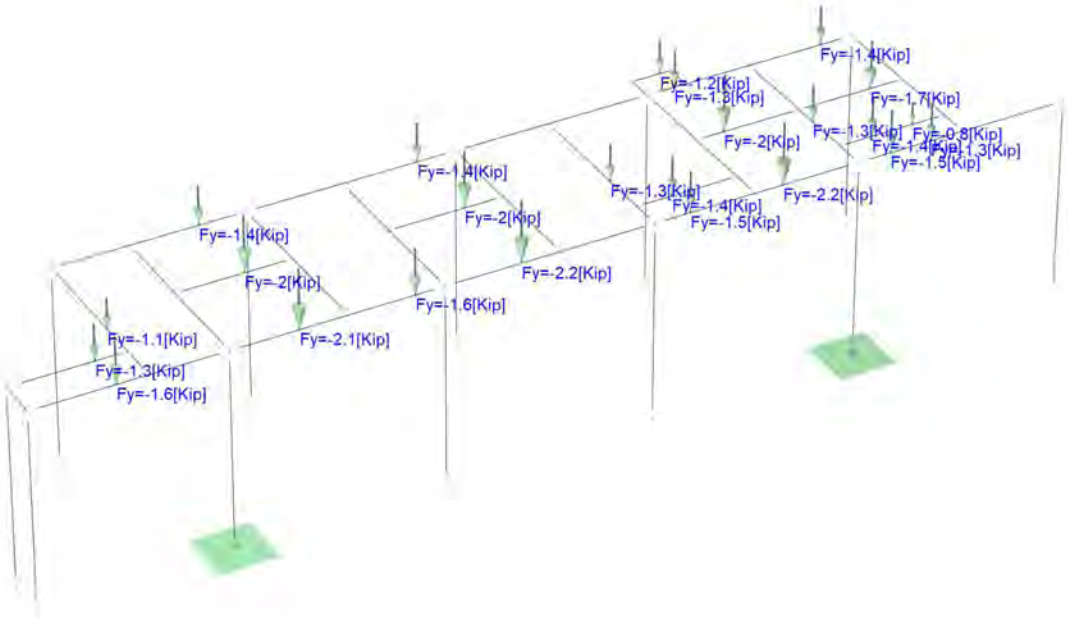


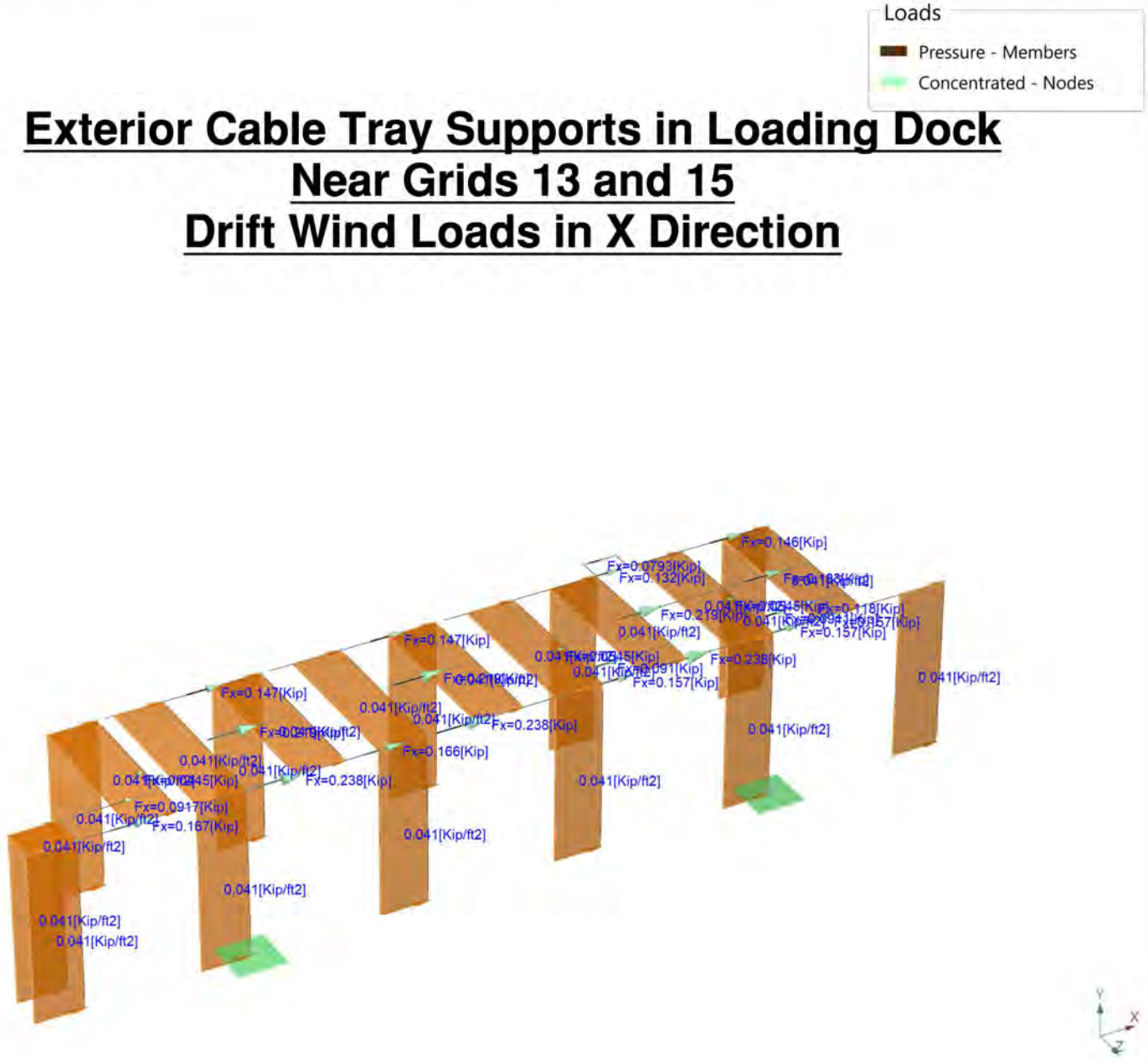
Exterior Cable Tray Supports in Loading Dock
Near Grids 13 and 15
Member Numbers



Loads
Concentrated - Nodes

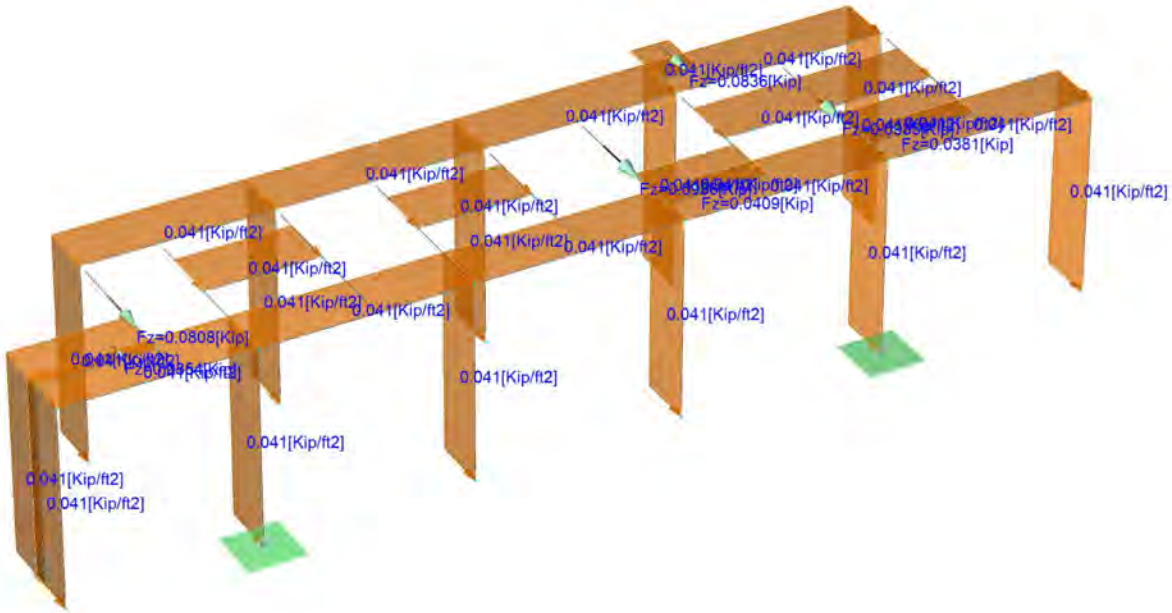
Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 Equipment Loads

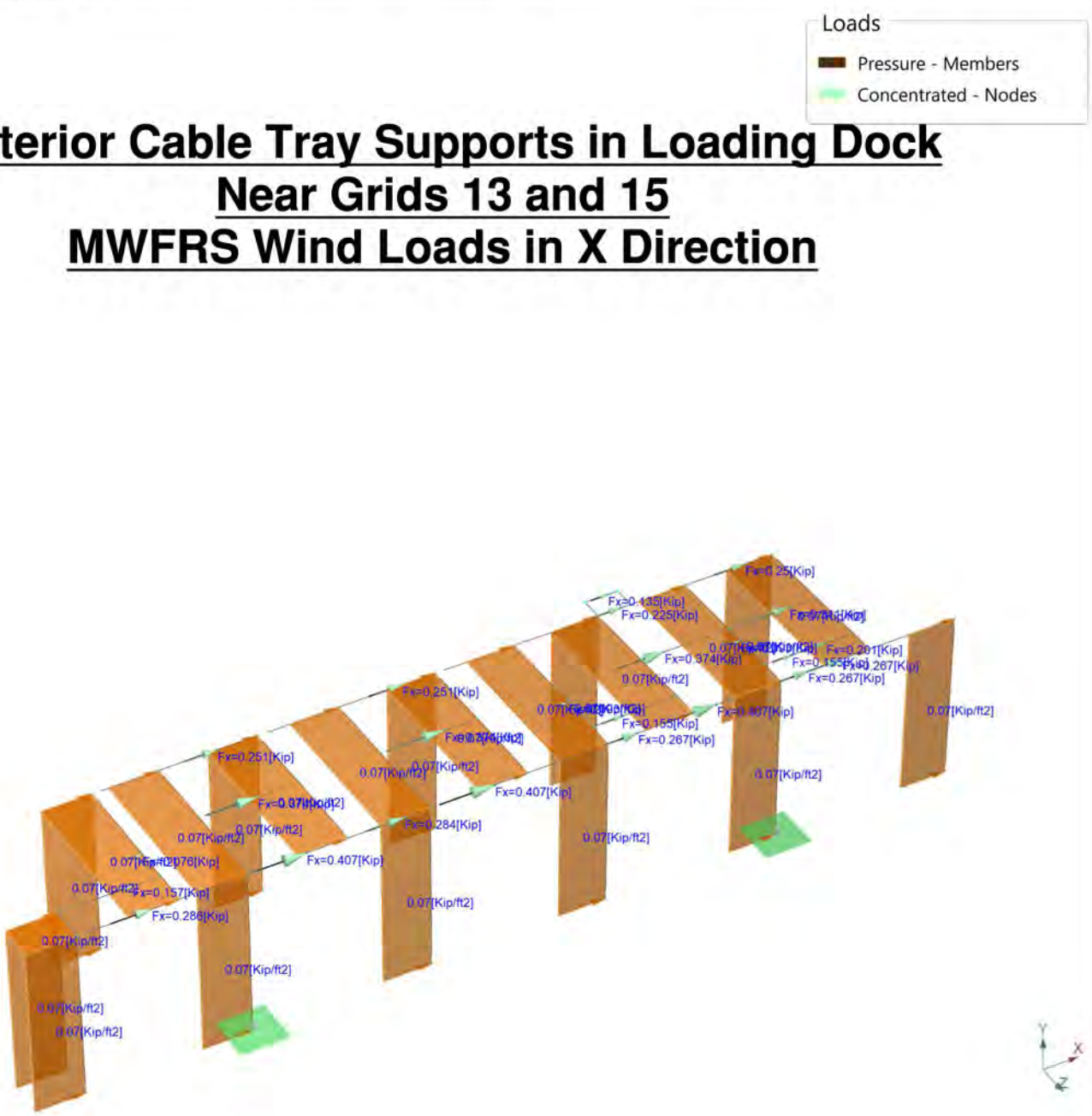




Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 Drift Wind Loads in X Direction

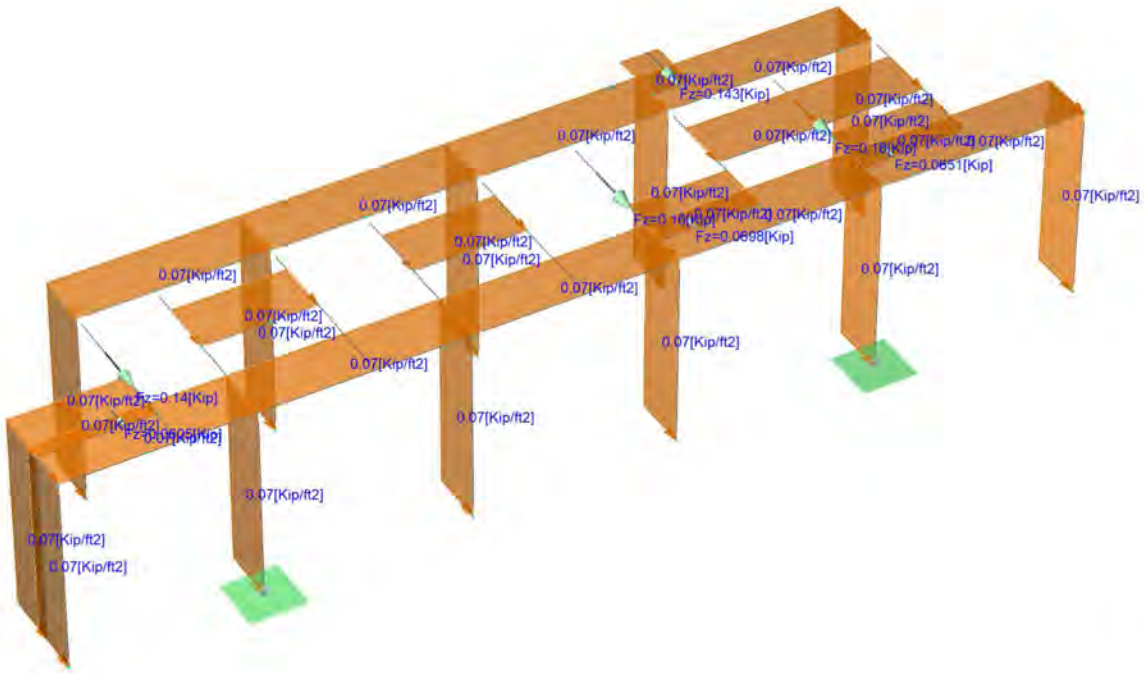
Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 Drift Wind Loads in Z Direction



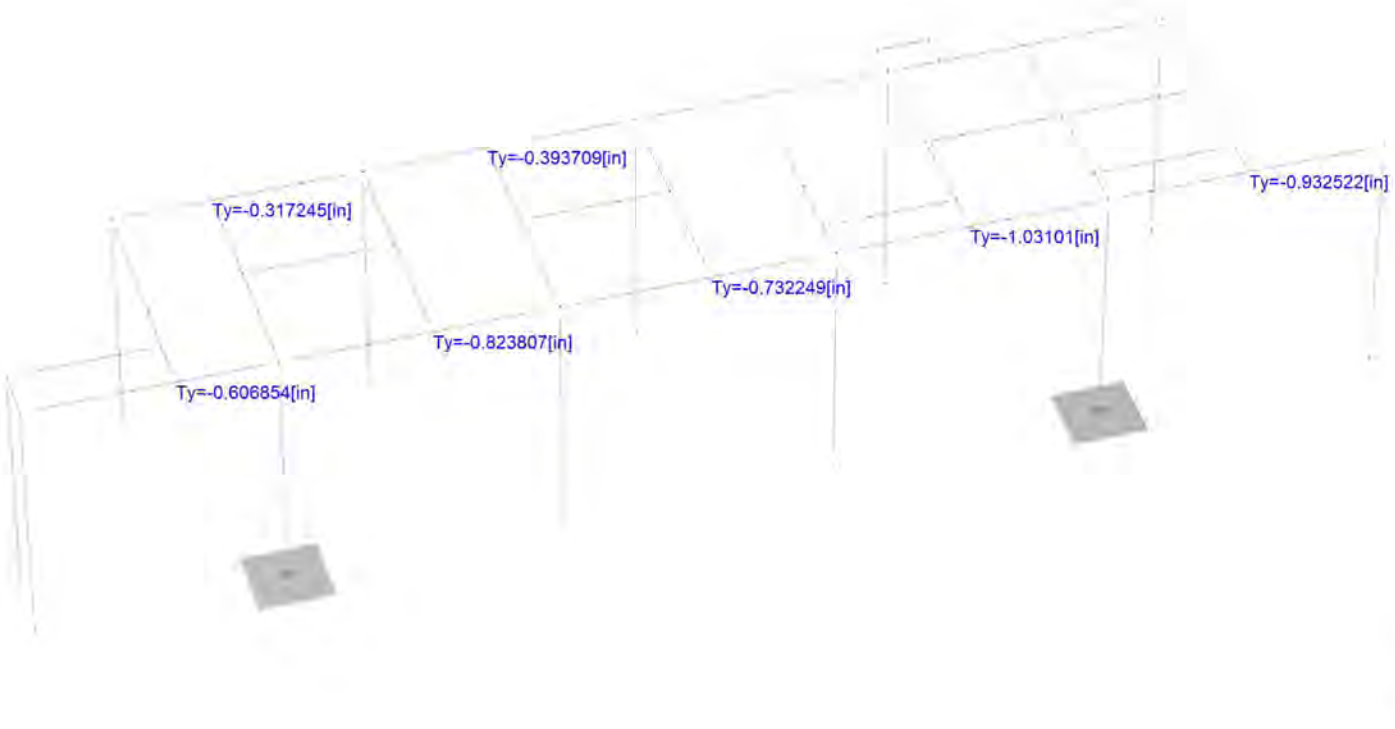


Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 MWFRS Wind Loads in X Direction

Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 MWFRS Wind Loads in Z Direction

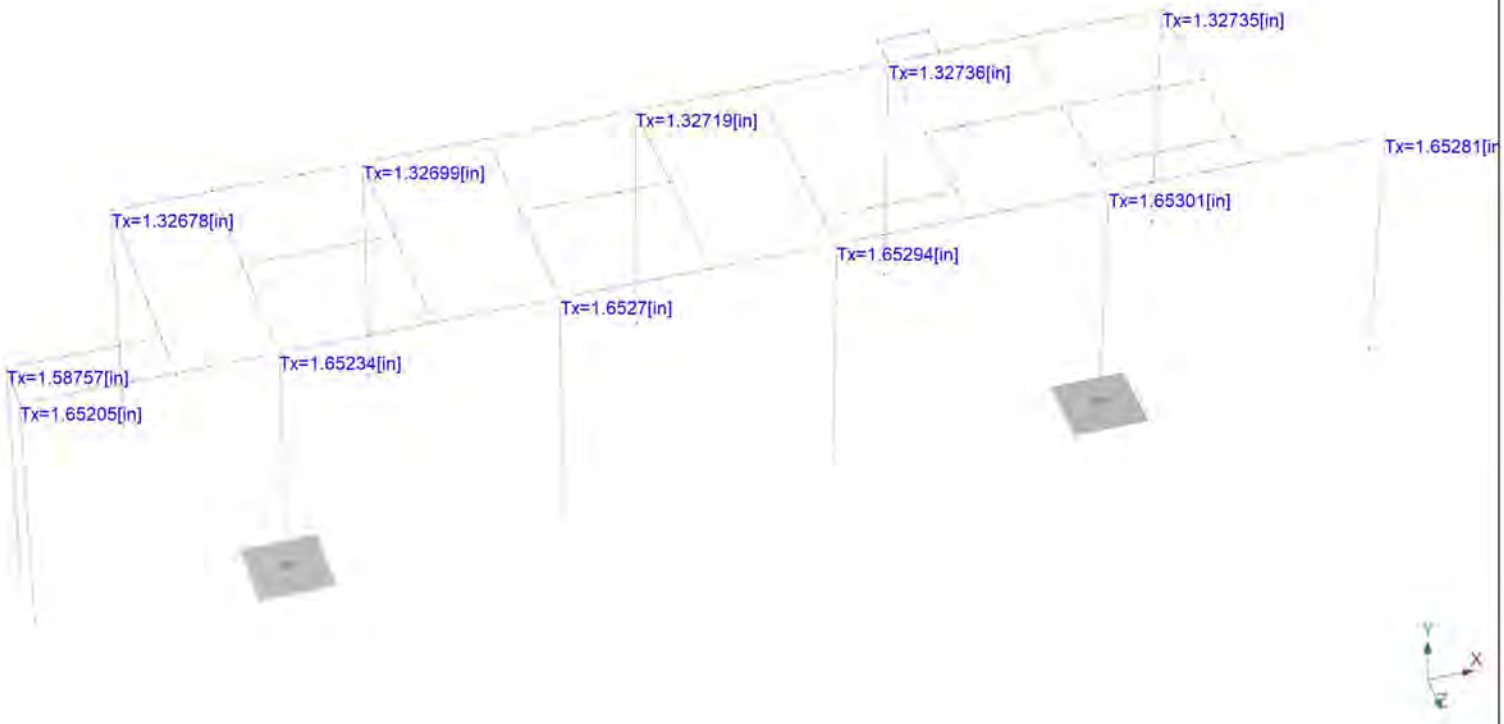


Exterior Cable Tray Supports in Loading Dock
Near Grids 13 and 15
Nodal Displacement for Equipment Weight



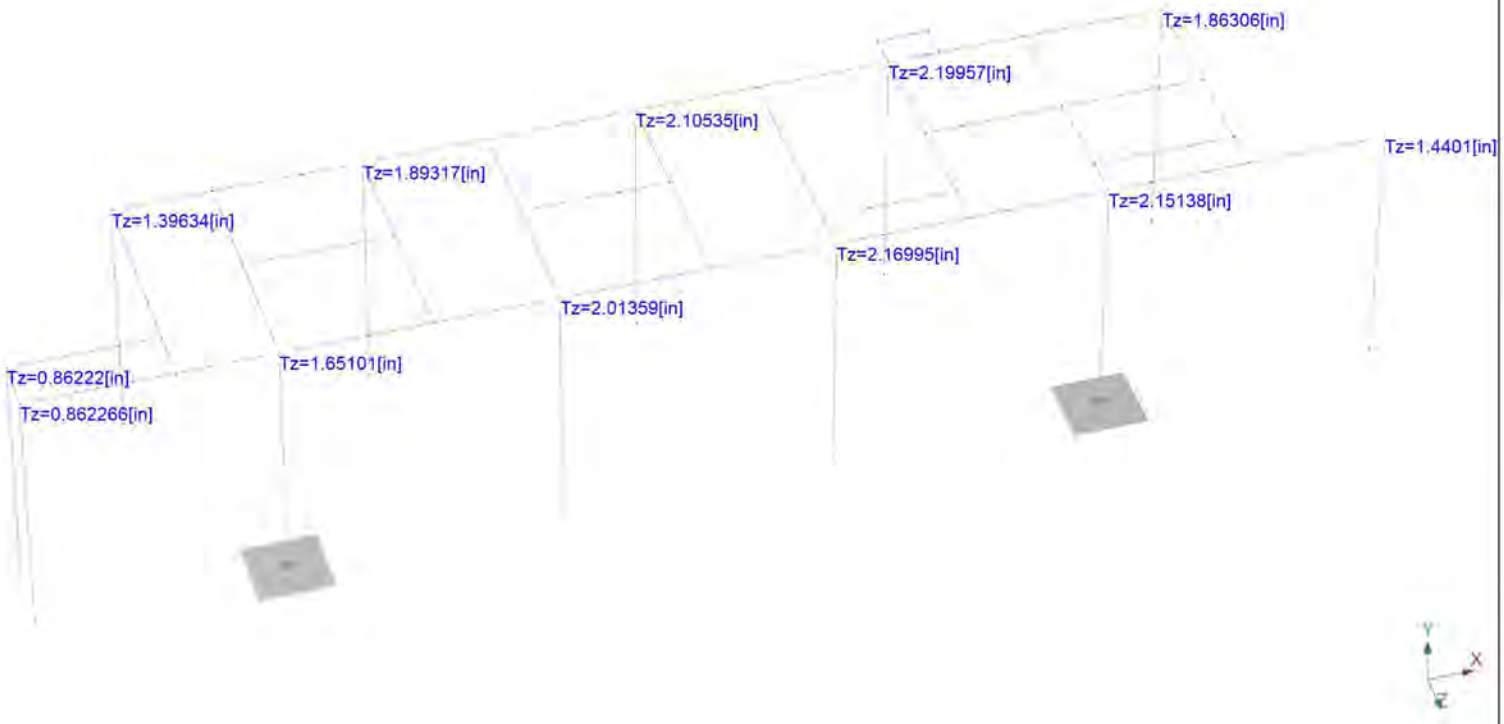
Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 Nodal Displacement for Drift Wind in X Direction

Note: Drift limits meet the L/120 limit since this is a flexible system



Exterior Cable Tray Supports in Loading Dock Near Grids 13 and 15 Nodal Displacement for Drift Wind in Z Direction

Note: Drift limits meet the L/120 limit since this is a flexible system



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:19 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Cable Tray Supports at Loading Dock\Cable Tray Support at Loading Dock between Grids 13 and 15.retx

Steel Code Check
Comprehensive Report

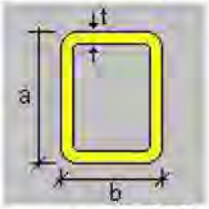
AISC 360-2016 LRFD (Hot-rolled)

Member : 5 (column) - OK

Section information

Section name: HSS_SQR 7X7X5_16 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	7.000	[in]	Width
T	=	0.291	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.590	
Moment of Inertia (local axes) (I)	[in4]	56.100	56.100
Moment of Inertia (principal axes) (I')	[in4]	56.100	56.100
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.719	2.719
Radius of gyration (principal axes) (r')	[in]	2.719	2.719
Saint-Venant torsion constant. (J)	[in4]	89.700	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	16.000	16.000
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	16.000	16.000
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	16.000	16.000
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	16.000	16.000
Plastic section modulus (local axis) (Z)	[in3]	18.900	18.900
Plastic section modulus (principal axis) (Z')	[in3]	18.900	18.900
Polar radius of gyration. (ro)	[in]	3.844	
Area for shear (Aw)	[in2]	3.566	3.566
Torsional constant. (C)	[in3]	26.099	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	21.67

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
21.67	21.67

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
21.67	21.67	21.67	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.55 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.55	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.08			
Capacity	:	175.00 [Kip]	Reference	:	Cl.E3
Demand	:	13.31 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	21.05	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	21.05	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	175.00	Cl.E3
Effective length (KL_{33})	[ft]	21.67	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	95.63	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	31.29	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	25.62	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	194.44	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.08			
Capacity	:	175.00 [Kip]	Reference	:	Cl.E3
Demand	:	13.31 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	21.05	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	21.05	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	175.00	Cl.E3
Effective length (KL_{22})	[ft]	21.67	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	95.63	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	31.29	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	25.62	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	194.44	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.48			
Capacity	:	70.88 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-33.77 [Kip*ft]	Ctrl Eq.	:	D14 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	
Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength(ϕM_n):	[Kip*ft]	70.88	Cl.F7.1
Yielding (M_n)	[Kip*ft]	78.75	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.64		
Capacity	:	70.88 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-45.14 [Kip*ft]	Ctrl Eq.	: D17 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	
Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	70.88	Cl.F7.1
Yielding (M_n)	[Kip*ft]	78.75	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.02		
Capacity	:	96.28 [Kip]	Reference	: Cl.G1
Demand	:	2.39 [Kip]	Ctrl Eq.	: D25 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	96.28	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	106.98	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.02		
Capacity	:	96.28 [Kip]	Reference	: Cl.G1
Demand	:	2.17 [Kip]	Ctrl Eq.	: D22 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	96.28	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	106.98	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	58.72 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	58.72	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	65.25	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.67			
Ctrl Eq.	:	D17 at 0.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.67	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	70.88	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	70.88	Cl.H1.1
Available axial strength (P_c)	[Kip]	175.00	Cl.H1.1

RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 1:20 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Cable Tray Supports at Loading Dock\Cable Tray Support at Loading Dock between Grids 13 and 15.retx

Steel Code Check
Comprehensive Report

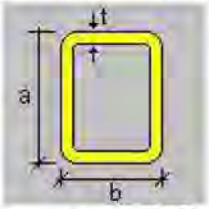
AISC 360-2016 LRFD (Hot-rolled)

Member : 15 (Girder) - OK

Section information

Section name: HSS_SQR 7X7X5_16 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	7.000	[in]	Width
T	=	0.291	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.590	
Moment of Inertia (local axes) (I)	[in4]	56.100	56.100
Moment of Inertia (principal axes) (I')	[in4]	56.100	56.100
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.719	2.719
Radius of gyration (principal axes) (r')	[in]	2.719	2.719
Saint-Venant torsion constant. (J)	[in4]	89.700	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	16.000	16.000
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	16.000	16.000
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	16.000	16.000
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	16.000	16.000
Plastic section modulus (local axis) (Z)	[in3]	18.900	18.900
Plastic section modulus (principal axis) (Z')	[in3]	18.900	18.900
Polar radius of gyration. (ro)	[in]	3.844	
Area for shear (Aw)	[in2]	3.566	3.566
Torsional constant. (C)	[in3]	26.099	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	22.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
22.00	22.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
22.00	22.00	22.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.55 [Kip]	Reference	:	Cl.D2
Demand	:	0.83 [Kip]	Ctrl Eq.	:	D14 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.55	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	171.41 [Kip]	Reference	:	Cl.E3
Demand	:	0.86 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	21.05	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	21.05	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	171.41	Cl.E3
Effective length (KL_{33})	[ft]	22.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	97.11	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	30.35	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	25.09	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	190.45	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	171.41 [Kip]	Reference	:	Cl.E3
Demand	:	0.86 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	21.05	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	21.05	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	171.41	Cl.E3
Effective length (KL_{22})	[ft]	22.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	97.11	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	30.35	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	25.09	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	190.45	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.40			
Capacity	:	70.88 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	28.20 [Kip*ft]	Ctrl Eq.	:	D1 at 48.86%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	70.88	Cl.F7.1
Yielding (M_n)	[Kip*ft]	78.75	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.08		
Capacity	:	70.88 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	5.39 [Kip*ft]	Ctrl Eq.	: D17 at 48.86%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	21.05	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	70.88	Cl.F7.1
Yielding (M_n)	[Kip*ft]	78.75	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.01		
Capacity	:	96.28 [Kip]	Reference	: Cl.G1
Demand	:	0.72 [Kip]	Ctrl Eq.	: D17 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	96.28	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	106.98	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.04		
Capacity	:	96.28 [Kip]	Reference	: Cl.G1
Demand	:	4.18 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	96.28	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	106.98	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	58.72 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.24 [Kip*ft]	Ctrl Eq.	:	D16 at 62.50%

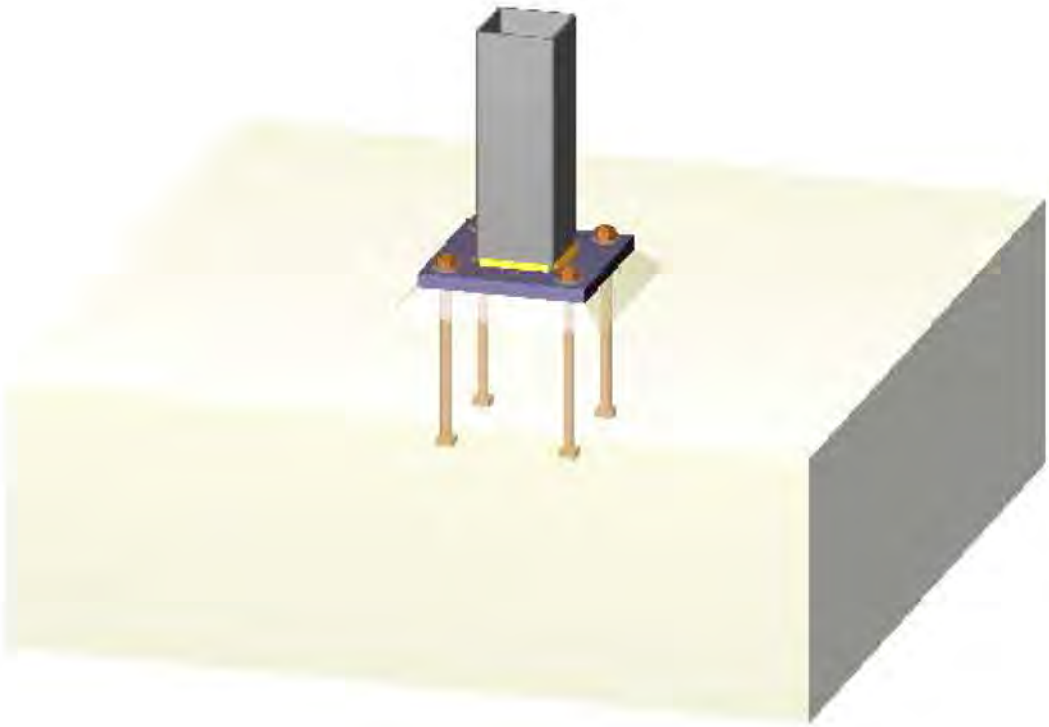
Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	58.72	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	65.25	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.42			
Ctrl Eq.	:	D17 at 48.86%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.42	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	70.88	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	70.88	Cl.H1.1
Available axial strength (P_c)	[Kip]	171.41	Cl.H1.1



Current Date: 8/17/2026 1:04 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Cable Tray Supports at Loading Dock\Cable Tray Support at Loading Dock between Grids 13 and 15.retx

Steel Connections Results

Connection: 1 - Fixed biaxial BP

Family:	Column - Base (CB)
Type:	Base plate
Description:	Smart Fixed Biaxial Base Plate 1

Design code: AISC 360-16 LRFD, ACI 318-19

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL	-9.50	1.22	0.18	-0.01	0.06	Design
W1	0.00	-0.04	18.69	-1.31	0.00	Design
W2	0.00	0.04	-18.69	1.31	0.00	Design
W3	0.00	-23.45	0.00	0.00	-1.41	Design
W4	0.00	23.44	0.00	0.00	1.41	Design
W5	-0.01	-0.06	31.92	-2.23	0.00	Design
W6	0.01	0.06	-31.92	2.23	0.00	Design
W7	0.00	-40.04	0.00	0.00	-2.41	Design
W8	0.00	40.02	0.00	0.00	2.41	Design
D1	-13.31	1.76	0.25	-0.01	0.08	Design
D2	-11.41	1.47	10.04	-0.64	0.07	Design
D3	-11.40	1.50	-9.61	0.61	0.07	Design
D4	-11.41	-11.30	0.21	-0.01	-0.61	Design
D5	-11.41	14.27	0.21	-0.01	0.74	Design
D6	-11.41	1.46	16.99	-1.08	0.07	Design
D7	-11.40	1.52	-16.57	1.06	0.07	Design
D8	-11.40	-20.35	0.21	-0.01	-1.09	Design
D9	-11.41	23.31	0.21	-0.01	1.22	Design
D10	-11.41	1.45	19.87	-1.26	0.07	Design
D11	-11.40	1.52	-19.44	1.24	0.07	Design
D12	-11.40	-24.09	0.21	-0.01	-1.29	Design
D13	-11.41	27.06	0.21	-0.01	1.42	Design
D14	-11.41	1.43	33.77	-2.15	0.06	Design
D15	-11.40	1.55	-33.35	2.12	0.07	Design
D16	-11.40	-42.18	0.21	-0.01	-2.24	Design
D17	-11.41	45.14	0.21	-0.01	2.38	Design
D18	-8.56	1.05	19.56	-1.27	0.05	Design
D19	-8.55	1.13	-19.24	1.26	0.05	Design
D20	-8.55	-23.91	0.16	-0.01	-1.32	Design
D21	-8.56	26.08	0.16	-0.01	1.42	Design
D22	-8.56	1.03	33.28	-2.17	0.05	Design
D23	-8.55	1.15	-32.97	2.15	0.06	Design
D24	-8.55	-41.59	0.16	-0.01	-2.29	Design
D25	-8.56	43.75	0.16	-0.01	2.39	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]	8	3	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	3.32	1.04	D17	0.31	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	12.66	8.80	D17	0.70	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	12.66	11.01	D17	0.87	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	200.46	50.72	D17	0.25	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	133.64	85.39	W6	0.64	Sec. J2.4
Ratio	**0.87**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	11.00	4.00	--	✔	Sec. 17.9.2
Concrete cover	[in]	30.00	3.00	--	✔	Sec. 20.5.1.3.1
Effective length	[in]	12.65	--	23.35	✔	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	26.35	21.13	D17	0.80	Eq. 17.6.1.2
Breakout of anchor in tension	[Kip]	38.58	21.13	D17	0.55	Sec. 17.5.2
Breakout of group of anchors in tension	[Kip]	50.32	42.12	D17	0.84	Sec. 17.5.2
Pullout of anchor in tension	[Kip]	24.61	21.13	D17	0.86	Sec. 17.5.2
Anchor shear	[Kip]	10.96	0.60	W7	0.05	Eq. 17.7.1.2b, Sec. 17.7.1.2.1
Breakout of anchor in shear	[Kip]	30.49	0.56	W6	0.02	Sec. 17.5.2
Breakout of anchor in shear parallel to edge	[Kip]	66.25	0.56	W6	0.01	Sec. 17.5.2
Breakout of group of anchors in shear	[Kip]	33.13	2.23	W6	0.07	Sec. 17.5.2
Breakout of group of anchors in shear parallel to edge	[Kip]	66.25	2.23	W6	0.03	Sec. 17.5.2
Pryout of anchor in shear	[Kip]	77.17	0.56	W6	0.01	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	131.53	2.23	W6	0.02	Sec. 17.5.2
Interaction of tensile and shear forces		1.20	0.00	DL	0.00	Sec. 17.8.1
Ratio	**0.86**					

Design for minor 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]	8	3	--	✓	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	3.32	1.04	D17	0.31	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	12.66	8.80	D17	0.70	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	12.66	11.01	D17	0.87	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	200.46	50.72	D17	0.25	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	133.64	115.16	D17	0.86	Sec. J2.4
Ratio	**0.87**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	11.00	4.00	--	✓	Sec. 17.9.2
Concrete cover	[in]	30.00	3.00	--	✓	Sec. 20.5.1.3.1
Effective length	[in]	12.65	--	23.35	✓	

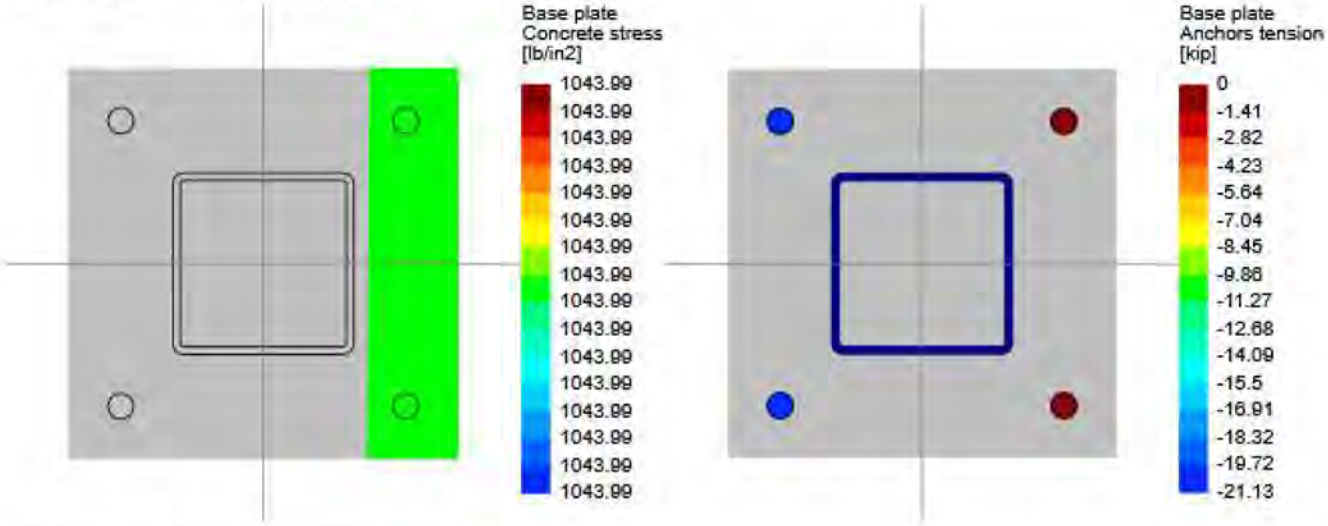
Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	26.35	21.13	D17	0.80	Eq. 17.6.1.2
Breakout of anchor in tension	[Kip]	38.58	21.13	D17	0.55	Sec. 17.5.2
Breakout of group of anchors in tension	[Kip]	50.32	42.12	D17	0.84	Sec. 17.5.2
Pullout of anchor in tension	[Kip]	24.61	21.13	D17	0.86	Sec. 17.5.2
Anchor shear	[Kip]	10.96	0.60	W7	0.05	Eq. 17.7.1.2b, Sec. 17.7.1.2.1
Breakout of anchor in shear	[Kip]	30.49	0.60	W7	0.02	Sec. 17.5.2
Breakout of anchor in shear parallel to edge	[Kip]	66.25	0.60	W7	0.01	Sec. 17.5.2
Breakout of group of anchors in shear	[Kip]	33.13	2.41	W7	0.07	Sec. 17.5.2
Breakout of group of anchors in shear parallel to edge	[Kip]	66.25	2.41	W7	0.04	Sec. 17.5.2
Pryout of anchor in shear	[Kip]	77.17	0.60	W7	0.01	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	131.53	2.41	W7	0.02	Sec. 17.5.2
Interaction of tensile and shear forces		1.20	0.00	DL	0.00	Sec. 17.8.1

Ratio	**0.86**
Global critical strength ratio	**0.87**

Biaxial

Maximum compression and tension (D17)



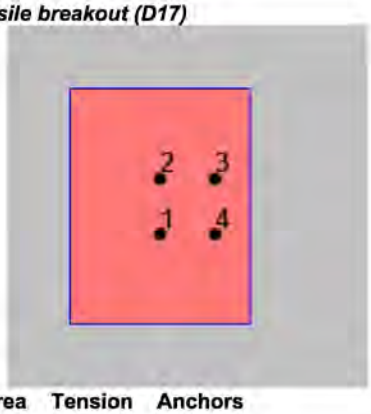
Maximum bearing pressure	1043.99	[lb/in2]
Minimum bearing pressure	1043.99	[lb/in2]
Maximum anchor tension	21.13	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	-90.35	[deg]
Neutral axis location	3.46	[in]
Bearing length	3.46	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-5.50	-5.50	0.00	20.99
2	-5.50	5.50	0.00	21.13
3	5.50	5.50	0.00	0.00
4	5.50	-5.50	0.00	0.00

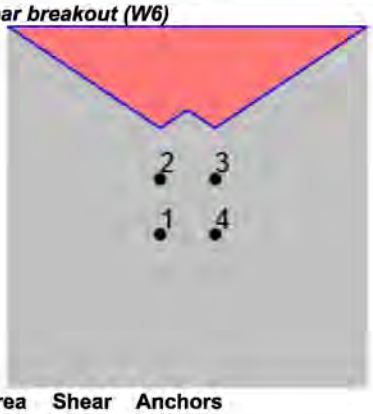
Major axis

Results for tensile breakout (D17)



Group	Area [in2]	Tension [kip]	Anchors
1	1692.00	42.12	1, 2

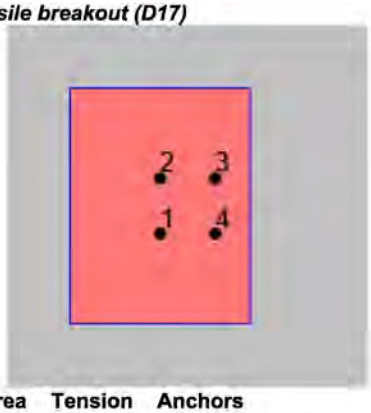
Results for shear breakout (W6)



Group	Area [in2]	Shear [kip]	Anchors
1	1728.00	2.23	1, 2, 3, 4
2	1728.00	1.12	2, 3

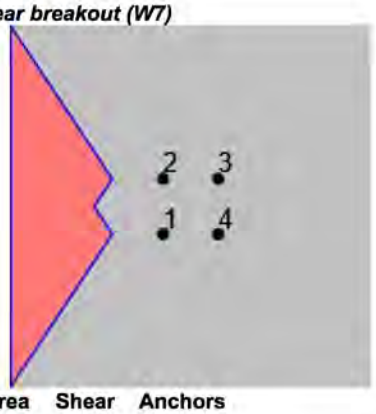
Minor axis

Results for tensile breakout (D17)

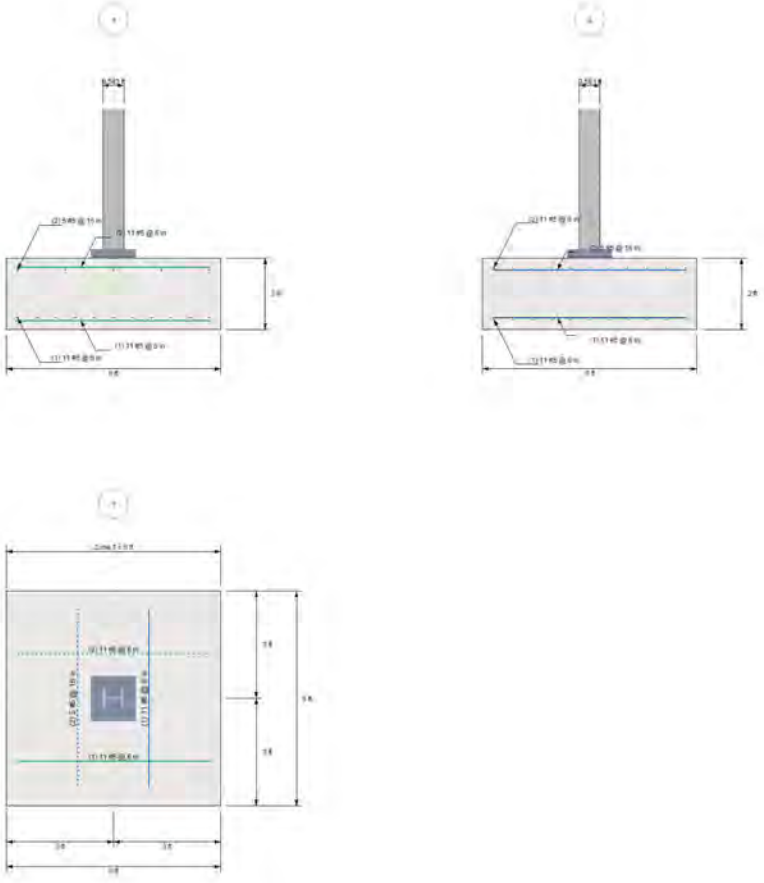


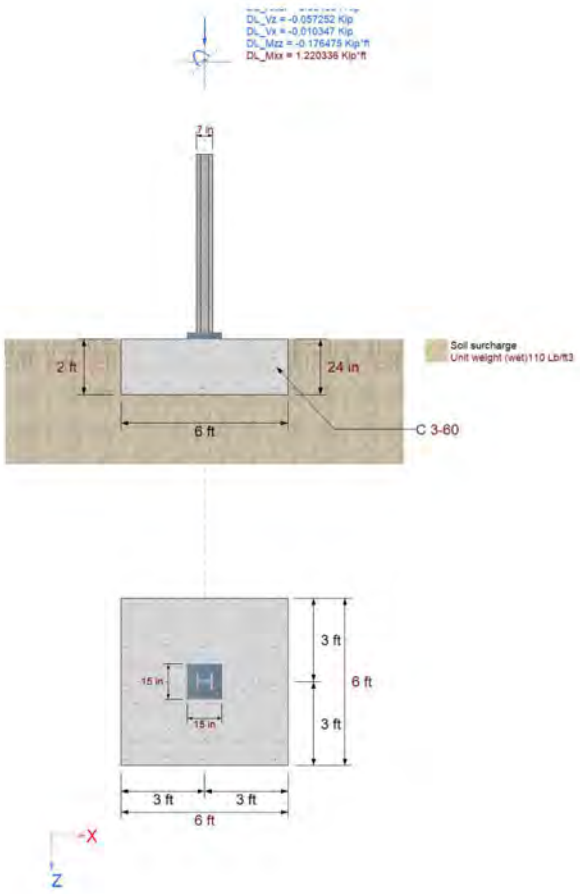
Group	Area [in2]	Tension [kip]	Anchors
1	1692.00	42.12	1, 2

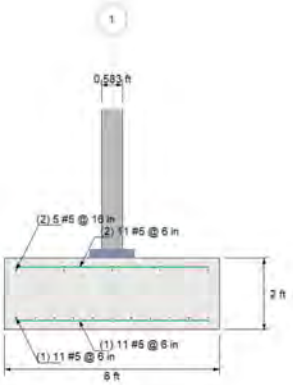
Results for shear breakout (W7)

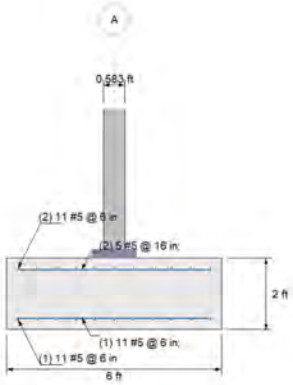


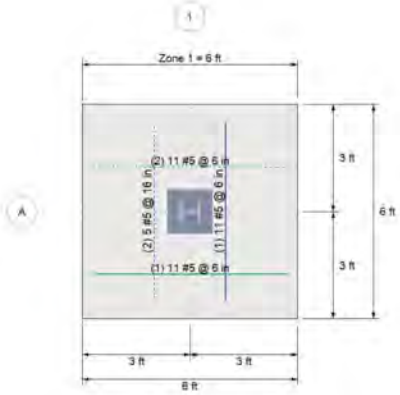
Group	Area [in2]	Shear [kip]	Anchors
1	1728.00	1.21	1, 2
2	1728.00	2.41	1, 2, 3, 4




DL_Vz = -0.057252 Kip
DL_Vx = -0.010347 Kip
DL_Mzz = -0.176475 Kip*ft
DL_Mxx = 1.220336 Kip*ft
7 in
2 ft
24 in
6 ft
Soil surcharge
Unit weight (wet)110 Lb/ft3
C 3-60
3 ft
6 ft
3 ft
15 in
15 in
3 ft
3 ft
6 ft
X
Z


1
0.583 ft
(2) 5 #5 @ 16 in
(2) 11 #5 @ 6 in
2 ft
(1) 11 #5 @ 6 in
(1) 11 #5 @ 6 in
8 ft


A
0.583 ft
(2) 11 #5 @ 6 in
(2) 5 #5 @ 16 in
2 ft
(1) 11 #5 @ 6 in
(1) 11 #5 @ 6 in
6 ft


1
Zone 1 = 6 ft
A
(2) 11 #5 @ 6 in
(2) 5 #5 @ 16 in
(1) 11 #5 @ 6 in
(1) 11 #5 @ 6 in
3 ft
3 ft
6 ft
3 ft
3 ft
6 ft

Design Results
Reinforced Concrete Footings

General Information

Global status	:	OK
Design Code	:	ACI 318-2019
Footing type	:	Spread
Column type	:	Steel

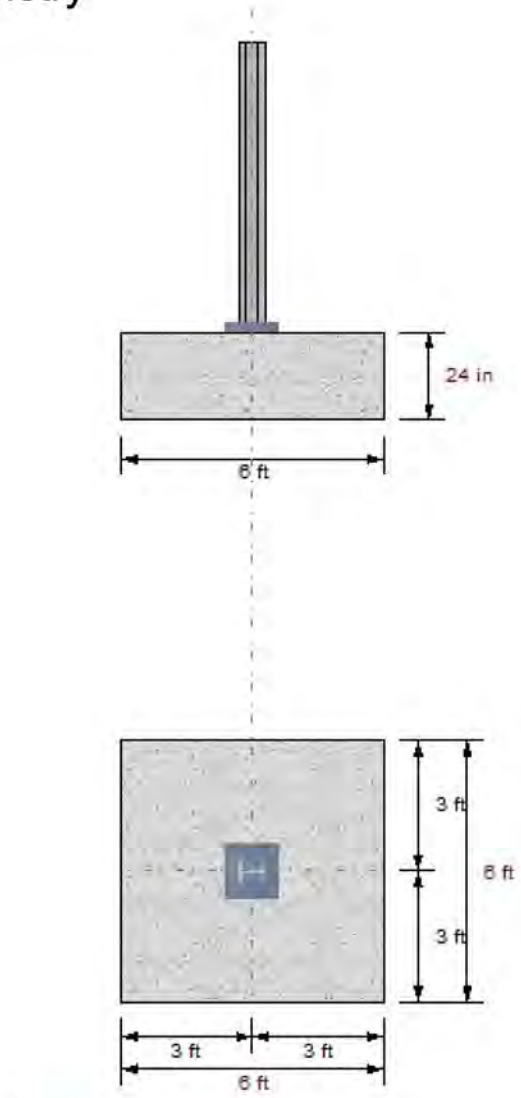
Materials

Concrete, f'c	:	3.00 [Kip/in2]	Steel, fy	:	60.00 [Kip/in2]
Concrete type	:	Normal	Epoxy coated	:	No
Concrete elasticity modulus	:	3122.02 [Kip/in2]	Steel elasticity modulus	:	29000.00 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]			

Soil

Modulus of subgrade reaction	:	200.00 [Kip/ft3]
Unit weight (wet)	:	0.11 [Kip/ft3]

Geometry



Length	:	6.00 [ft]
Width	:	6.00 [ft]
Thickness	:	2.00 [ft]
Base depth	:	2.00 [ft]
Base area	:	36.00 [ft2]
Footing volume	:	72.00 [ft3]
Base plate length	:	15.00 [in]
Base plate width	:	15.00 [in]
Column length	:	7.00 [in]
Column width	:	7.00 [in]
Column location relative to footing g.c.	:	Centered

Reinforcement

Longitudinal reinforcement

Free cover	:	3.00 [in]
Maximum Rho/Rho balanced ratio	:	0.75
Bottom reinforcement // to L (xx)	:	11-#5 @ 6.00"
Top reinforcement // to L (xx)	:	11-#5 @ 6.00"
Bottom reinforcement // to B (zz)	:	11-#5 @ 6.00" (Zone 1)
Top reinforcement // to B (zz)	:	5-#5 @ 16.00"

Load Conditions

Service loads:

S1	:	_S1
S2	:	_S2
S3	:	_S3
S4	:	_S4
S5	:	_S5
S6	:	_S6
S7	:	_S7
S8	:	_S8
S9	:	_S9
S10	:	_S10
S11	:	_S11
S12	:	_S12
S13	:	_S13
S14	:	_S14
S15	:	_S15
S16	:	_S16
S17	:	_S17

Design strength loads:

D1	:	_D1
D2	:	_D2
D3	:	_D3
D4	:	_D4
D5	:	_D5
D6	:	_D6
D7	:	_D7
D8	:	_D8
D9	:	_D9
D10	:	_D10
D11	:	_D11
D12	:	_D12
D13	:	_D13
D14	:	_D14
D15	:	_D15
D16	:	_D16
D17	:	_D17
D18	:	_D18
D19	:	_D19
D20	:	_D20
D21	:	_D21
D22	:	_D22
D23	:	_D23
D24	:	_D24
D25	:	_D25

Loads

Condition	Footing	Node	Axial [Kip]	Mxx [Kip*ft]	Mzz [Kip*ft]	Vx [Kip]	Vz [Kip]
DL	1	7	9.50	1.22	-0.18	-0.01	-0.06
W1	1	7	0.00	-0.04	-18.69	-1.31	0.00
W2	1	7	0.00	0.04	18.69	1.31	0.00
W3	1	7	0.00	-23.45	0.00	0.00	1.41
W4	1	7	0.00	23.44	0.00	0.00	-1.41
W5	1	7	0.01	-0.06	-31.92	-2.23	0.00
W6	1	7	-0.01	0.06	31.92	2.23	0.00
W7	1	7	0.00	-40.04	0.00	0.00	2.41
W8	1	7	0.00	40.02	0.00	0.00	-2.41
_D1	1	7	13.31	1.76	-0.25	-0.01	-0.08
_D2	1	7	11.41	1.47	-10.04	-0.64	-0.07
_D3	1	7	11.40	1.50	9.61	0.61	-0.07
_D4	1	7	11.41	-11.30	-0.21	-0.01	0.61
_D5	1	7	11.41	14.27	-0.21	-0.01	-0.74
_D6	1	7	11.41	1.46	-16.99	-1.08	-0.07
_D7	1	7	11.40	1.52	16.57	1.06	-0.07
_D8	1	7	11.40	-20.35	-0.21	-0.01	1.09
_D9	1	7	11.41	23.31	-0.21	-0.01	-1.22
_D10	1	7	11.41	1.45	-19.87	-1.26	-0.07
_D11	1	7	11.40	1.52	19.44	1.24	-0.07
_D12	1	7	11.40	-24.09	-0.21	-0.01	1.29
_D13	1	7	11.41	27.06	-0.21	-0.01	-1.42
_D14	1	7	11.41	1.43	-33.77	-2.15	-0.06
_D15	1	7	11.40	1.55	33.35	2.12	-0.07
_D16	1	7	11.40	-42.18	-0.21	-0.01	2.24
_D17	1	7	11.41	45.14	-0.21	-0.01	-2.38
_D18	1	7	8.56	1.05	-19.56	-1.27	-0.05
_D19	1	7	8.55	1.13	19.24	1.26	-0.05
_D20	1	7	8.55	-23.91	-0.16	-0.01	1.32
_D21	1	7	8.56	26.08	-0.16	-0.01	-1.42
_D22	1	7	8.56	1.03	-33.28	-2.17	-0.05
_D23	1	7	8.55	1.15	32.97	2.15	-0.06
_D24	1	7	8.55	-41.59	-0.16	-0.01	2.29
_D25	1	7	8.56	43.75	-0.16	-0.01	-2.39
_S1	1	7	9.50	1.22	-0.18	-0.01	-0.06
_S2	1	7	9.51	1.20	-11.87	-0.77	-0.06
_S3	1	7	9.50	1.24	11.51	0.75	-0.06
_S4	1	7	9.50	-13.89	-0.18	-0.01	0.76
_S5	1	7	9.51	16.33	-0.18	-0.01	-0.88
_S6	1	7	9.51	1.18	-20.14	-1.30	-0.06
_S7	1	7	9.50	1.26	19.79	1.28	-0.06
_S8	1	7	9.50	-24.58	-0.18	-0.01	1.34
_S9	1	7	9.51	27.01	-0.18	-0.01	-1.45
_S10	1	7	5.71	0.69	-11.60	-0.77	-0.03
_S11	1	7	5.70	0.73	11.39	0.76	-0.04
_S12	1	7	5.70	-13.96	-0.10	-0.01	0.80
_S13	1	7	5.70	15.38	-0.10	-0.01	-0.87
_S14	1	7	5.71	0.67	-19.73	-1.32	-0.03
_S15	1	7	5.70	0.75	19.52	1.31	-0.04
_S16	1	7	5.70	-24.34	-0.10	-0.01	1.38
_S17	1	7	5.70	25.75	-0.10	-0.01	-1.45

Design

Status : OK

Soil Foundation Interaction

Allowable stress	:	3.5E03 [Lb/ft2]
Min. safety factor for sliding	:	1.50
Min. safety factor for overturning	:	1.50

Controlling condition : S17 - 1

Condition	qmean	qmax	Δmax	Area in compression		Overturning		FS
Footing	[Lb/ft2]	[Lb/ft2]	[in]	[ft2]	(%)	FSx	FSz	slip
S17 - 1	726	1.46E03	0.0874	22.72	63	1.73	424.42	5.78

Bending

Factor ϕ	:	0.90
Min rebar ratio	:	0.00180

Development length

Axis	Pos.	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
z	Bot.	24.98	12.60	27.50	27.50
x	Bot.	24.98	12.60	27.50	27.50
z	Top	12.00	12.60	29.50	29.50
x	Top	12.00	12.60	29.50	29.50

Axis	Pos.	Condition Footing	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	D22 - 1	-5.81	-308.90	0.06	3.41	0.018	0.019
zz	Bot.	D14 - 1	23.22	308.90	3.05	3.41	0.894	0.075
xx	Top	D16 - 1	-5.81	-138.17	0.06	1.55	0.042	0.042
xx	Bot.	D17 - 1	33.91	299.31	3.05	3.41	0.893	0.113

Shear

Factor ϕ	:	0.75
Shear area (plane zz)	:	10.34 [ft2]
Shear area (plane xx)	:	10.03 [ft2]

Plane	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	D25 - 1	14.53	84.28	0.230
yz	D22 - 1	7.23	86.02	0.112

Punching shear

Factor ϕ	:	0.75
Perimeter of critical section (b...	:	10.46 [ft]
Punching shear area	:	17.76 [ft2]

Column	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)	
column 1	D17 - 1	12.85	560.22	0.031	

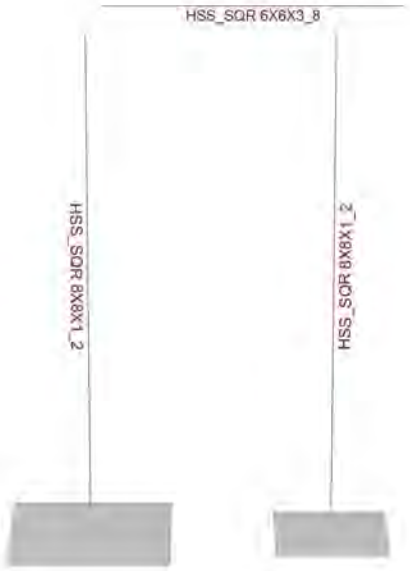
Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed.
* The required flexural reinforcement considers at least the minimum reinforcement
* The design bending moment is calculated at the critical sections located at the support faces
* Only rectangular footings with uniform sections and rectangular columns are considered.
* The nominal shear strength is calculated in critical sections located at a distance d from the support face
* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces
* Transverse reinforcement is not considered in footings
* Values shown in red are not in compliance with a provision of the code
*qprom = Mean compression pressure on soil.
*qmax = Maximum compression pressure on soil.
*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus).
* Mn = Nominal moment strength
* Mu/(ϕ*Mn) = Strength ratio.
* Vn = Nominal shear or punchure force (for footings Vn=Vc).
* Vu/(ϕ*Vn) = Shear or punching shear strength ratio.

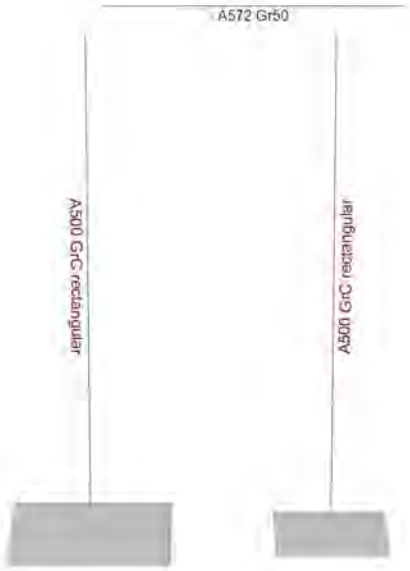
Exterior H-Framing with Cantilever
3D Rendering



Exterior H-Framing Member Sizes

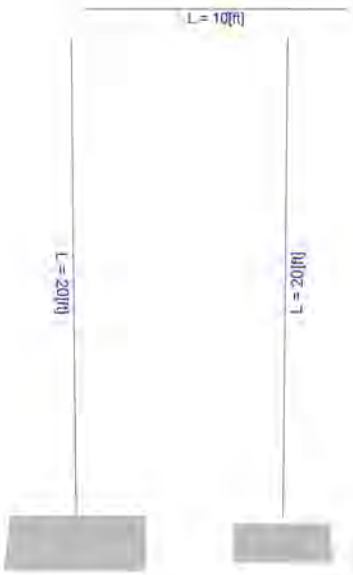


Exterior H-Framing Material

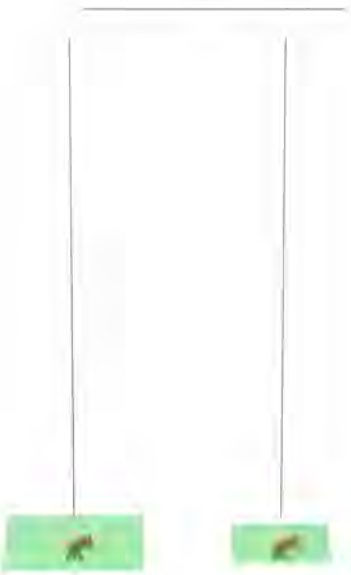




Exterior H-Framing Lengths

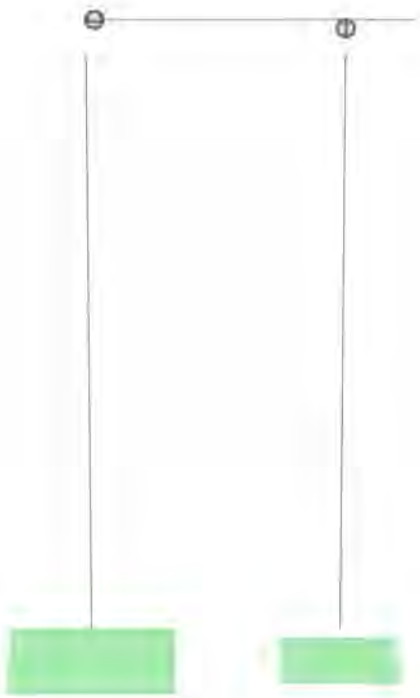


Exterior H-Framing Supports



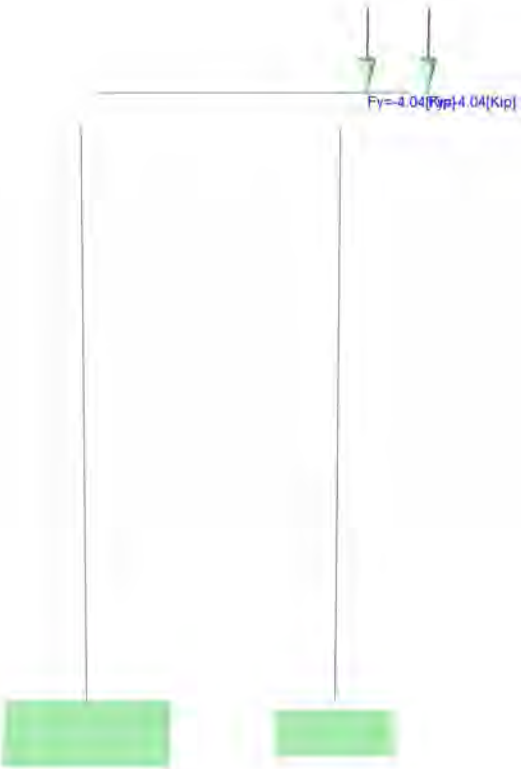


Exterior H-Framing Hinges



Loads
Concentrated - Nodes

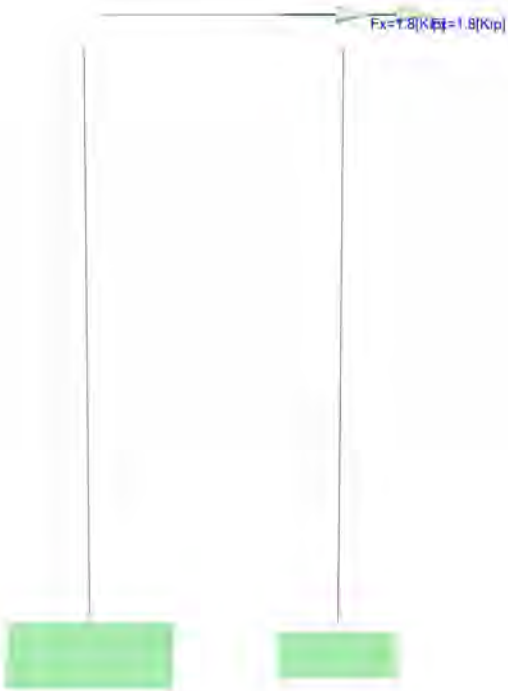
Exterior H-Framing Dead Loads





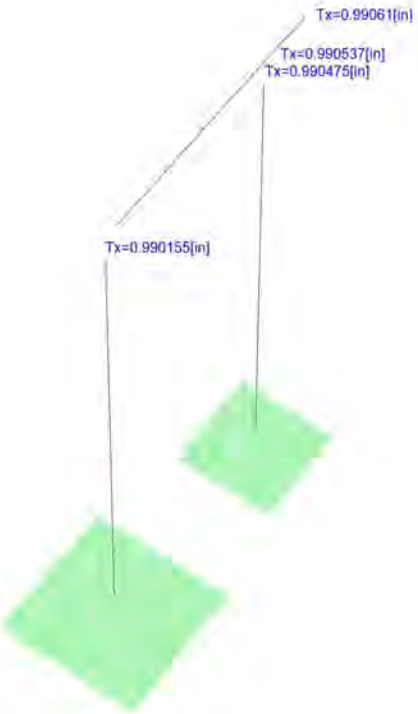
Loads
Concentrated - Nodes

Exterior H-Framing Wind Loads in X-Direction



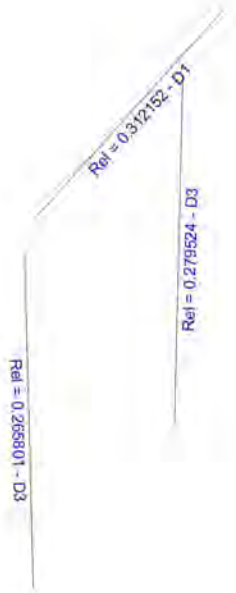


Exterior H-Framing Deflections





Exterior H-Framing Stress Ratios





RAM Elements **Salas O'Brien**

Current Date: 9/21/2026 6:22 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pipe Supports BLDG 1\Exterior\Cantilevered H-Frame\h-fRAME - Exterior Rev 01.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5WLx
D3=1.2DL+WLx
D4=0.9DL+WLx

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_SQR 6X6X3_8	**3**	D1 at 75.00%	**0.31**	**OK**	
Post	***HSS_SQR 8X8X1_2***	**1**	D3 at 0.00%	0.27	OK	
		2	D3 at 0.00%	**0.28**	**OK**	

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: Equipment Foundation Support.ec6

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : ACI 318-19
Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength		3.0 ksi
fy : Rebar Yield		60.0 ksi
Ec : Concrete Elastic Modulus		3,122.0 ksi
Concrete Density		145.0 pcf
ϕ : Phi Values	Flexure :	0.90
	Shear :	0.750

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel Ratio per ACI 7.6.1.1 (ratio=Steel Area/Concrete Area)	0.0018
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	3.50 ksf
Increase Bearing By Footing Weight	No
Soil Passive Sliding Resistance	250.0 pcf
(Uses entry for "Footing base depth below soil surface" for force)	
Coefficient of Soil/Concrete Friction	0.30

Soil Bearing Increase

Footing base depth below soil surface	ft
Increases based on footing Depth	
Allowable pressure increase per foot	ksf
when base of footing is below	ft
Increases based on footing Width . . .	
Allowable pressure increase per foot	ksf
when maximum length or width is greater tha	ft
Maximum Allowed Bearing Pressure	10.0 ksf
(A value of zero implies no limit)	
Adjusted Allowable Soil Bearing	3.50 ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

Distance Left of Column #1	=	1.50 ft
Between Columns	=	8.0 ft
Distance Right of Column #2	=	1.50 ft
Total Footing Length	=	11.0 ft
Footing Width	=	4.0 ft
Footing Thickness	=	24.0 in

Pedestal dimensions...

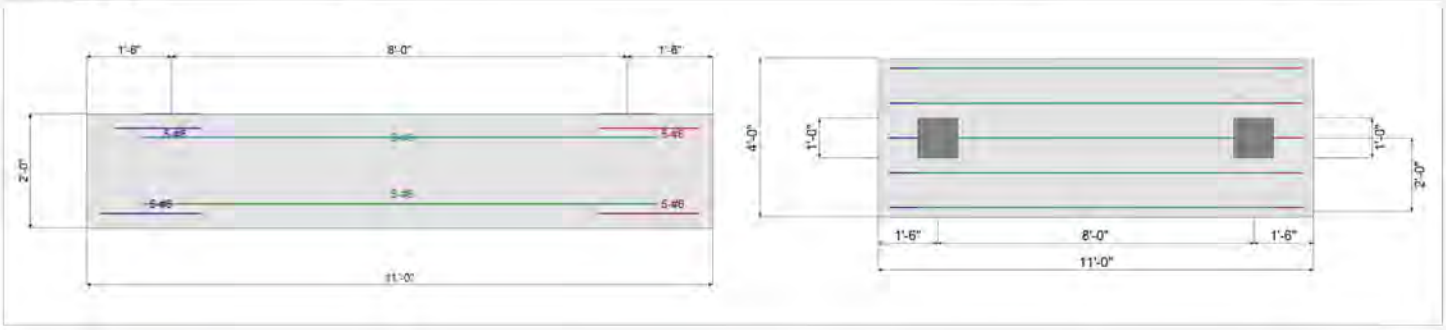
	Col #1	Col #2
Sq. Dim. =	12.0	12.0 in
Height =		in

Rebar Center to Concrete Edge @ Top	=	3.0 in	
Rebar Center to Concrete Edge @ Bottom	=	3.0 in	

Bars left of Col #1	Count	Size #	As Provided
Bottom Bars	5.0	6	2.20
Top Bars	5.0	6	2.20
Bars Btwn Cols			
Bottom Bars	5.0	6	2.20
Top Bars	5.0	6	2.20
Bars Right of Col #2			
Bottom Bars	5.0	6	2.20
Top Bars	5.0	6	2.20

Applied Loads

Applied @ Left Column		D	Lr	L	S	W	E	H
Axial Load Downward	=	1.0						k
Moment (+CW)	=					30.590		k-ft
Shear (+X)	=					1.80		k
Applied @ Right Column								
Axial Load Downward	=	10.0						k
Moment (+CW)	=					30.590		k-ft
Shear (+X)	=					1.80		k
Overburden	=							



Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: Equipment Foundation Support.ec6

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

DESIGN SUMMARY

Design OK

Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS 1.385	Overturning	41.028 k-ft	56.808 k-ft	+0.60D+0.60W
PASS 1.980	Sliding	2.160 k	4.277 k	+0.60D+0.60W
PASS No Uplift	Uplift	0.0 k	0.0 k	No Uplift

Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS 0.6108	Soil Bearing	2.138 ksf	3.50 ksf	+0.60D+0.60W
PASS 0.1494	1-way Shear - Col #1	6.369 psi	42.628 psi	+1.20D+W
PASS 0.2323	1-way Shear - Col #2	9.904 psi	42.628 psi	+1.20D+W
PASS 0.008412	2-way Punching - Col #1	1.382 psi	164.317 psi	+1.20D+W
PASS 0.02543	2-way Punching - Col #2	4.178 psi	164.317 psi	+1.20D+W
PASS 0.003368	Flexure - Left of Col #1 - Top	-0.6822 k-ft	202.562 k-ft	+1.20D+W
PASS No Tension	Flexure - Left of Col #1 - Bottom	0.0 k-ft	0.0 k-ft	N/A
PASS 0.1103	Flexure - Between Cols - Top	-22.338 k-ft	202.562 k-ft	+1.20D+W
PASS 0.1561	Flexure - Between Cols - Bottom	31.629 k-ft	202.562 k-ft	+0.90D+W
PASS No Tension	Flexure - Right of Col #2 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS 0.03517	Flexure - Right of Col #2 - Bottom	7.123 k-ft	202.562 k-ft	+0.90D+W

Soil Bearing

Load Combination...	Total Bearing	Eccentricity from Ftg CL	Actual Soil Bearing Stress @ Left Edge	@ Right Edge	Allowable	Actual / Allow Ratio
D Only	23.76 k	1.515 ft	0.09 ksf	0.99 ksf	3.50 ksf	0.281
+D+0.60W	23.76 k	3.242 ft	0.00 ksf	1.75 ksf	3.50 ksf	0.500
+D+0.450W	23.76 k	2.810 ft	0.00 ksf	1.47 ksf	3.50 ksf	0.420
+0.60D+0.60W	14.26 k	4.393 ft	0.00 ksf	2.14 ksf	3.50 ksf	0.611
+0.60D	14.26 k	1.515 ft	0.06 ksf	0.59 ksf	3.50 ksf	0.169

Overturning Stability

Load Combination...	Moments about Left Edge k-ft Overturning	Resisting	Ratio	Moments about Right Edg k-ft Overturning	Resisting	Ratio
D Only	0.00	0.00	999.000	0.00	0.00	999.000
+D+0.60W	0.00	0.00	999.000	41.03	94.68	2.308
+D+0.450W	0.00	0.00	999.000	30.77	94.68	3.077
+0.60D+0.60W	0.00	0.00	999.000	41.03	56.81	1.385
+0.60D	0.00	0.00	999.000	0.00	0.00	999.000

Sliding Stability

Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio
D Only	0.00 k	7.13 k	999
+D+0.60W	2.16 k	7.13 k	3.3
+D+0.450W	1.62 k	7.13 k	4.4
+0.60D+0.60W	2.16 k	4.28 k	1.98
+0.60D	0.00 k	4.28 k	999

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.60D	0.000	0.000	0	0.000	0	0.000	0.000	0.000
+0.60D	0.000	0.028	0	0.000	0	0.000	0.000	0.000
+0.60D	0.000	0.055	0	0.000	0	0.000	0.000	0.000
+0.60D	0.000	0.083	0	0.000	0	0.000	0.000	0.000
+0.60D	0.000	0.110	0	0.000	0	0.000	0.000	0.000
+1.20D+W	-0.013	0.138	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.019	0.165	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.026	0.193	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.034	0.220	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.043	0.248	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.053	0.275	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.064	0.303	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+W	-0.076	0.330	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.089	0.358	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+1.20D+W	-0.103	0.385	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.118	0.413	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.135	0.440	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.152	0.468	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.171	0.495	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.190	0.523	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.211	0.550	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.232	0.578	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.255	0.605	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.278	0.633	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.303	0.660	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+1.20D+W	-0.329	0.688	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.356	0.715	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.384	0.743	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.413	0.770	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.443	0.798	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.474	0.825	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.506	0.853	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+1.20D+W	-0.539	0.880	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+1.20D+W	-0.573	0.908	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+1.20D+W	-0.609	0.935	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+1.20D+W	-0.645	0.963	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+1.20D+W	-0.682	0.990	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+1.20D+W	-0.721	1.018	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+1.20D+W	-0.761	1.045	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+1.20D+W	-0.804	1.073	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+1.20D+W	-0.848	1.100	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+1.20D+W	-0.895	1.128	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+1.20D+W	-0.943	1.155	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.005
+1.20D+W	-0.993	1.183	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.005
+1.20D+W	-1.046	1.210	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.005
+1.20D+W	-1.100	1.238	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.005
+1.20D+W	-1.156	1.265	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.006
+1.20D+W	-1.214	1.293	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.006
+1.20D+W	-1.274	1.320	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.006
+1.20D+W	-1.336	1.348	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.007
+1.20D+W	-1.400	1.375	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.007
+1.20D+W	-1.466	1.403	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.007
+1.20D+W	-1.534	1.430	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.008
+1.20D+W	-1.604	1.458	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.008
+1.20D+W	-1.676	1.485	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.008
+0.90D+W	32.878	1.513	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.162
+0.90D+W	32.821	1.540	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.162
+0.90D+W	32.762	1.568	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.162
+0.90D+W	32.703	1.595	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.161
+0.90D+W	32.641	1.623	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.161
+0.90D+W	32.579	1.650	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.161
+0.90D+W	32.514	1.678	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.161
+0.90D+W	32.449	1.705	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.160
+0.90D+W	32.382	1.733	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.160
+0.90D+W	32.313	1.760	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.160
+0.90D+W	32.243	1.788	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.159
+0.90D+W	32.171	1.815	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.159
+0.90D+W	32.098	1.843	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.158
+0.90D+W	32.024	1.870	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.158
+0.90D+W	31.948	1.898	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.158
+0.90D+W	31.870	1.925	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.157
+0.90D+W	31.792	1.953	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.157
+0.90D+W	31.711	1.980	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.157
+0.90D+W	31.629	2.008	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.156
+0.90D+W	31.547	2.035	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.156

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	31.463	2.063	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.155
+0.90D+W	31.379	2.090	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.155
+0.90D+W	31.294	2.118	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.154
+0.90D+W	31.208	2.145	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.154
+0.90D+W	31.121	2.173	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.154
+0.90D+W	31.033	2.200	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.153
+0.90D+W	30.945	2.228	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.153
+0.90D+W	30.856	2.255	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.152
+0.90D+W	30.766	2.283	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.152
+0.90D+W	30.675	2.310	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.151
+0.90D+W	30.584	2.338	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.151
+0.90D+W	30.492	2.365	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.151
+0.90D+W	30.399	2.392	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.150
+0.90D+W	30.305	2.420	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.150
+0.90D+W	30.210	2.447	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.149
+0.90D+W	30.115	2.475	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.149
+0.90D+W	30.018	2.502	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.148
+0.90D+W	29.921	2.530	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.148
+0.90D+W	29.824	2.557	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.147
+0.90D+W	29.725	2.585	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.147
+0.90D+W	29.626	2.612	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.146
+0.90D+W	29.526	2.640	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.146
+0.90D+W	29.425	2.667	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.145
+0.90D+W	29.323	2.695	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.145
+0.90D+W	29.220	2.722	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.144
+0.90D+W	29.117	2.750	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.144
+0.90D+W	29.013	2.777	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.143
+0.90D+W	28.908	2.805	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.143
+0.90D+W	28.802	2.832	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.142
+0.90D+W	28.696	2.860	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.142
+0.90D+W	28.589	2.887	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.141
+0.90D+W	28.481	2.915	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.141
+0.90D+W	28.372	2.942	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.140
+0.90D+W	28.262	2.970	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.140
+0.90D+W	28.152	2.997	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.139
+0.90D+W	28.041	3.025	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.138
+0.90D+W	27.929	3.052	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.138
+0.90D+W	27.816	3.080	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.137
+0.90D+W	27.702	3.107	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.137
+0.90D+W	27.588	3.135	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.136
+0.90D+W	27.473	3.162	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.136
+0.90D+W	27.357	3.190	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.135
+0.90D+W	27.240	3.217	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.134
+0.90D+W	27.123	3.245	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.134
+0.90D+W	27.004	3.272	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.133
+0.90D+W	26.885	3.300	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.133
+0.90D+W	26.765	3.327	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.132
+0.90D+W	26.645	3.355	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.132
+0.90D+W	26.523	3.382	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.131
+0.90D+W	26.401	3.410	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.130
+0.90D+W	26.278	3.437	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.130
+0.90D+W	26.154	3.465	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.129
+0.90D+W	26.029	3.492	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.129
+0.90D+W	25.904	3.520	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.128
+0.90D+W	25.778	3.547	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.127
+0.90D+W	25.651	3.575	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.127
+0.90D+W	25.523	3.602	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.126
+0.90D+W	25.394	3.630	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.125
+0.90D+W	25.265	3.657	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.125
+0.90D+W	25.135	3.685	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.124
+0.90D+W	25.004	3.712	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.123
+0.90D+W	24.872	3.740	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.123
+0.90D+W	24.740	3.767	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.122

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	24.606	3.795	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.121
+0.90D+W	24.472	3.822	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.121
+0.90D+W	24.337	3.850	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.120
+0.90D+W	24.202	3.877	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.119
+0.90D+W	24.065	3.905	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.119
+0.90D+W	23.928	3.932	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.118
+0.90D+W	23.790	3.960	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.117
+0.90D+W	23.651	3.987	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.117
+0.90D+W	23.511	4.015	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.116
+0.90D+W	23.371	4.042	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.115
+0.90D+W	23.230	4.070	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.115
+0.90D+W	23.088	4.097	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.114
+0.90D+W	22.945	4.125	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.113
+0.90D+W	22.801	4.152	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.113
+0.90D+W	22.657	4.180	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.112
+0.90D+W	22.512	4.207	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.111
+0.90D+W	22.366	4.235	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.110
+0.90D+W	22.219	4.262	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.110
+0.90D+W	22.072	4.290	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.109
+0.90D+W	21.923	4.317	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.108
+0.90D+W	21.774	4.345	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.107
+0.90D+W	21.624	4.372	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.107
+0.90D+W	21.474	4.400	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.106
+0.90D+W	21.322	4.427	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.105
+0.90D+W	21.170	4.455	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.105
+0.90D+W	21.017	4.482	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.104
+0.90D+W	20.863	4.510	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.103
+0.90D+W	20.708	4.537	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.102
+0.90D+W	20.553	4.565	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.101
+0.90D+W	20.397	4.592	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.101
+0.90D+W	20.240	4.620	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.100
+0.90D+W	20.082	4.647	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.099
+0.90D+W	19.923	4.675	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.098
+0.90D+W	19.764	4.702	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.098
+0.90D+W	19.604	4.730	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.097
+0.90D+W	19.443	4.757	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.096
+0.90D+W	19.281	4.785	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.095
+0.90D+W	19.119	4.812	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.094
+0.90D+W	18.955	4.840	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.094
+0.90D+W	18.791	4.867	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.093
+0.90D+W	18.626	4.895	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.092
+0.90D+W	18.461	4.922	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.091
+0.90D+W	18.294	4.950	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.090
+0.90D+W	18.127	4.977	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.089
+0.90D+W	17.959	5.005	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.089
+0.90D+W	17.790	5.032	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.088
+0.90D+W	17.620	5.060	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.087
+0.90D+W	17.450	5.087	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.086
+0.90D+W	17.279	5.115	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.085
+0.90D+W	17.107	5.142	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.084
+0.90D+W	16.934	5.170	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.084
+0.90D+W	16.760	5.197	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.083
+0.90D+W	16.586	5.225	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.082
+0.90D+W	16.411	5.252	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.081
+0.90D+W	16.235	5.280	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.080
+0.90D+W	16.058	5.307	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.079
+0.90D+W	15.881	5.335	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.078
+0.90D+W	15.702	5.362	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.078
+0.90D+W	15.523	5.390	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.077
+0.90D+W	15.343	5.417	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.076
+0.90D+W	15.163	5.445	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.075
+0.90D+W	14.981	5.472	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.074
+0.90D+W	14.799	5.500	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.073

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	14.616	5.527	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.072
+0.90D+W	14.432	5.555	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.071
+0.90D+W	14.247	5.582	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.070
+0.90D+W	14.062	5.610	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+0.90D+W	13.876	5.637	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+0.90D+W	13.689	5.665	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.068
+0.90D+W	13.501	5.692	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.067
+0.90D+W	13.312	5.720	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.066
+0.90D+W	13.123	5.747	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.065
+0.90D+W	12.933	5.775	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.064
+0.90D+W	12.742	5.802	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.063
+0.90D+W	12.550	5.830	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.062
+0.90D+W	12.358	5.857	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.061
+0.90D+W	12.164	5.885	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.060
+0.90D+W	11.970	5.912	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.059
+0.90D+W	11.775	5.940	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.058
+0.90D+W	11.580	5.967	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.057
+0.90D+W	11.383	5.995	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.056
+0.90D+W	11.186	6.022	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.055
+0.90D+W	10.988	6.050	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.054
+0.90D+W	10.789	6.077	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.053
+0.90D+W	10.589	6.105	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.052
+0.90D+W	10.389	6.132	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.051
+0.90D+W	10.188	6.160	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.050
+0.90D+W	9.986	6.187	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.049
+0.90D+W	9.783	6.215	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.048
+1.40D	-9.633	6.242	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.048
+1.40D	-9.616	6.270	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.047
+1.20D+0.50W	-9.622	6.297	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.048
+1.20D+0.50W	-9.746	6.325	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.048
+1.20D+0.50W	-9.868	6.352	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.049
+1.20D+0.50W	-9.989	6.380	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.049
+1.20D+0.50W	-10.109	6.407	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.050
+1.20D+0.50W	-10.227	6.435	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.050
+1.20D+0.50W	-10.345	6.462	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.051
+1.20D+0.50W	-10.460	6.490	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.052
+1.20D+0.50W	-10.575	6.517	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.052
+1.20D+0.50W	-10.688	6.545	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.053
+1.20D+0.50W	-10.800	6.572	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.053
+1.20D+0.50W	-10.911	6.600	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.054
+1.20D+0.50W	-11.020	6.627	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.054
+1.20D+0.50W	-11.128	6.655	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.055
+1.20D+0.50W	-11.234	6.682	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.055
+1.20D+0.50W	-11.339	6.710	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.056
+1.20D+0.50W	-11.442	6.737	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.056
+1.20D+0.50W	-11.544	6.765	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.057
+1.20D+0.50W	-11.645	6.792	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.057
+1.20D+0.50W	-11.744	6.820	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.058
+1.20D+0.50W	-11.841	6.847	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.058
+1.20D+0.50W	-11.937	6.875	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.059
+1.20D+0.50W	-12.031	6.902	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.059
+1.20D+0.50W	-12.124	6.930	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.060
+1.20D+0.50W	-12.215	6.957	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.060
+1.20D+0.50W	-12.305	6.985	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.061
+1.20D+0.50W	-12.393	7.012	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.061
+1.20D+0.50W	-12.479	7.040	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.062
+1.20D+0.50W	-12.563	7.067	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.062
+1.20D+0.50W	-12.646	7.095	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.062
+1.20D+0.50W	-12.728	7.122	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.063
+1.20D+0.50W	-12.807	7.150	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.063
+1.20D+0.50W	-12.885	7.177	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.064
+1.20D+0.50W	-12.961	7.205	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.064
+1.20D+0.50W	-13.035	7.232	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.064

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50W	-13.108	7.260	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.065
+1.20D+0.50W	-13.178	7.287	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.065
+1.20D+0.50W	-13.247	7.315	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.065
+1.20D+0.50W	-13.314	7.342	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.066
+1.20D+0.50W	-13.379	7.370	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.066
+1.20D+0.50W	-13.443	7.397	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.066
+1.20D+0.50W	-13.504	7.425	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.067
+1.20D+0.50W	-13.564	7.452	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.067
+1.20D+0.50W	-13.621	7.480	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.067
+1.20D+0.50W	-13.677	7.507	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.068
+1.20D+0.50W	-13.731	7.535	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.068
+1.20D+0.50W	-13.783	7.562	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.068
+1.20D+0.50W	-13.832	7.590	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.068
+1.20D+0.50W	-13.880	7.617	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+1.20D+0.50W	-13.926	7.645	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+1.20D+0.50W	-13.970	7.672	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+1.20D+0.50W	-14.011	7.700	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+1.20D+0.50W	-14.051	7.727	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.069
+1.20D+0.50W	-14.088	7.755	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.070
+1.20D+0.50W	-14.124	7.782	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.070
+1.20D+W	-14.213	7.810	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.070
+1.20D+W	-14.459	7.837	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.071
+1.20D+W	-14.702	7.865	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.073
+1.20D+W	-14.944	7.892	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.074
+1.20D+W	-15.184	7.920	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.075
+1.20D+W	-15.421	7.947	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.076
+1.20D+W	-15.656	7.975	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.077
+1.20D+W	-15.889	8.002	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.078
+1.20D+W	-16.120	8.030	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.080
+1.20D+W	-16.349	8.057	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.081
+1.20D+W	-16.574	8.085	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.082
+1.20D+W	-16.798	8.112	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.083
+1.20D+W	-17.019	8.140	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.084
+1.20D+W	-17.237	8.167	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.085
+1.20D+W	-17.453	8.195	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.086
+1.20D+W	-17.666	8.222	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.087
+1.20D+W	-17.877	8.250	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.088
+1.20D+W	-18.084	8.277	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.089
+1.20D+W	-18.289	8.305	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.090
+1.20D+W	-18.491	8.332	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.091
+1.20D+W	-18.690	8.360	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.092
+1.20D+W	-18.886	8.387	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.093
+1.20D+W	-19.079	8.415	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.094
+1.20D+W	-19.269	8.442	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.095
+1.20D+W	-19.456	8.470	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.096
+1.20D+W	-19.639	8.497	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.097
+1.20D+W	-19.819	8.525	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.098
+1.20D+W	-19.997	8.552	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.099
+1.20D+W	-20.170	8.580	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.100
+1.20D+W	-20.340	8.607	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.100
+1.20D+W	-20.507	8.635	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.101
+1.20D+W	-20.671	8.662	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.102
+1.20D+W	-20.830	8.690	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.103
+1.20D+W	-20.986	8.717	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.104
+1.20D+W	-21.139	8.745	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.104
+1.20D+W	-21.288	8.772	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.105
+1.20D+W	-21.433	8.800	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.106
+1.20D+W	-21.574	8.827	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.107
+1.20D+W	-21.711	8.855	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.107
+1.20D+W	-21.845	8.882	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.108
+1.20D+W	-21.974	8.910	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.108
+1.20D+W	-22.099	8.937	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.109
+1.20D+W	-22.221	8.965	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.110

Combined Footing

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+W	-22.338	8.992	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.110
+1.20D+W	-22.453	9.020	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.111
+1.20D+W	-22.573	9.047	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.111
+1.20D+W	-22.698	9.075	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.112
+1.20D+W	-22.827	9.102	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.113
+1.20D+W	-22.961	9.130	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.113
+1.20D+W	-23.100	9.157	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.114
+1.20D+W	-23.243	9.185	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.115
+1.20D+W	-23.391	9.212	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.115
+1.20D+W	-23.543	9.240	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.116
+1.20D+W	-23.700	9.267	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.117
+1.20D+W	-23.861	9.295	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.118
+1.20D+W	-24.026	9.322	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.119
+1.20D+W	-24.196	9.350	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.119
+1.20D+W	-24.370	9.377	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.120
+1.20D+W	-24.548	9.405	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.121
+1.20D+W	-24.730	9.432	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.122
+1.20D+W	-24.916	9.460	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.123
+1.20D+W	-25.106	9.487	Top	2.074	ACI 7.6.1.1	2.200	202.562	0.124
+0.90D+W	13.574	9.515	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.067
+0.90D+W	13.228	9.542	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.065
+0.90D+W	12.880	9.570	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.064
+0.90D+W	12.530	9.597	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.062
+0.90D+W	12.178	9.625	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.060
+0.90D+W	11.824	9.652	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.058
+0.90D+W	11.468	9.680	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.057
+0.90D+W	11.110	9.707	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.055
+0.90D+W	10.752	9.735	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.053
+0.90D+W	10.392	9.762	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.051
+0.90D+W	10.031	9.790	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.050
+0.90D+W	9.669	9.817	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.048
+0.90D+W	9.306	9.845	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.046
+0.90D+W	8.943	9.872	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.044
+0.90D+W	8.579	9.900	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.042
+0.90D+W	8.215	9.927	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.041
+0.90D+W	7.851	9.955	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.039
+0.90D+W	7.487	9.982	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.037
+0.90D+W	7.123	10.010	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.035
+0.90D+W	6.766	10.038	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.033
+0.90D+W	6.415	10.065	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.032
+0.90D+W	6.072	10.093	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.030
+0.90D+W	5.737	10.120	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.028
+0.90D+W	5.410	10.148	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.027
+0.90D+W	5.090	10.175	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.025
+0.90D+W	4.779	10.203	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.024
+0.90D+W	4.476	10.230	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.022
+0.90D+W	4.181	10.258	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.021
+0.90D+W	3.895	10.285	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.019
+0.90D+W	3.618	10.313	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.018
+0.90D+W	3.350	10.340	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.017
+0.90D+W	3.090	10.368	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.015
+0.90D+W	2.840	10.395	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.014
+0.90D+W	2.600	10.423	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.013
+0.90D+W	2.369	10.450	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.012
+0.90D+W	2.147	10.478	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.011
+0.90D+W	1.936	10.505	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.010
+0.90D+W	1.734	10.533	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.009
+0.90D+W	1.543	10.560	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.008
+0.90D+W	1.362	10.588	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.007
+0.90D+W	1.192	10.615	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.006
+0.90D+W	1.032	10.643	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.005
+0.90D+W	0.883	10.670	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.004
+0.90D+W	0.745	10.698	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.004

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: Equipment Foundation Support.ec6

LIC# : KW-06018138, Build:20.26.08.18 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Cantilevered H-Frame Foundation

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	0.619	10.725	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.003
+0.90D+W	0.503	10.753	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+0.90D+W	0.399	10.780	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+0.90D+W	0.307	10.808	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.002
+0.90D+W	0.227	10.835	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+0.90D+W	0.158	10.863	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+0.90D+W	0.102	10.890	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.001
+0.90D+W	0.057	10.918	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+0.90D+W	0.026	10.945	Bottom	2.074	ACI 7.6.1.1	2.200	202.562	0.000
+0.90D+W	0.000	10.973	0	0.000	0	0.000	0.000	0.000
+0.90D+W	0.000	11.000	0	0.000	0	0.000	0.000	0.000

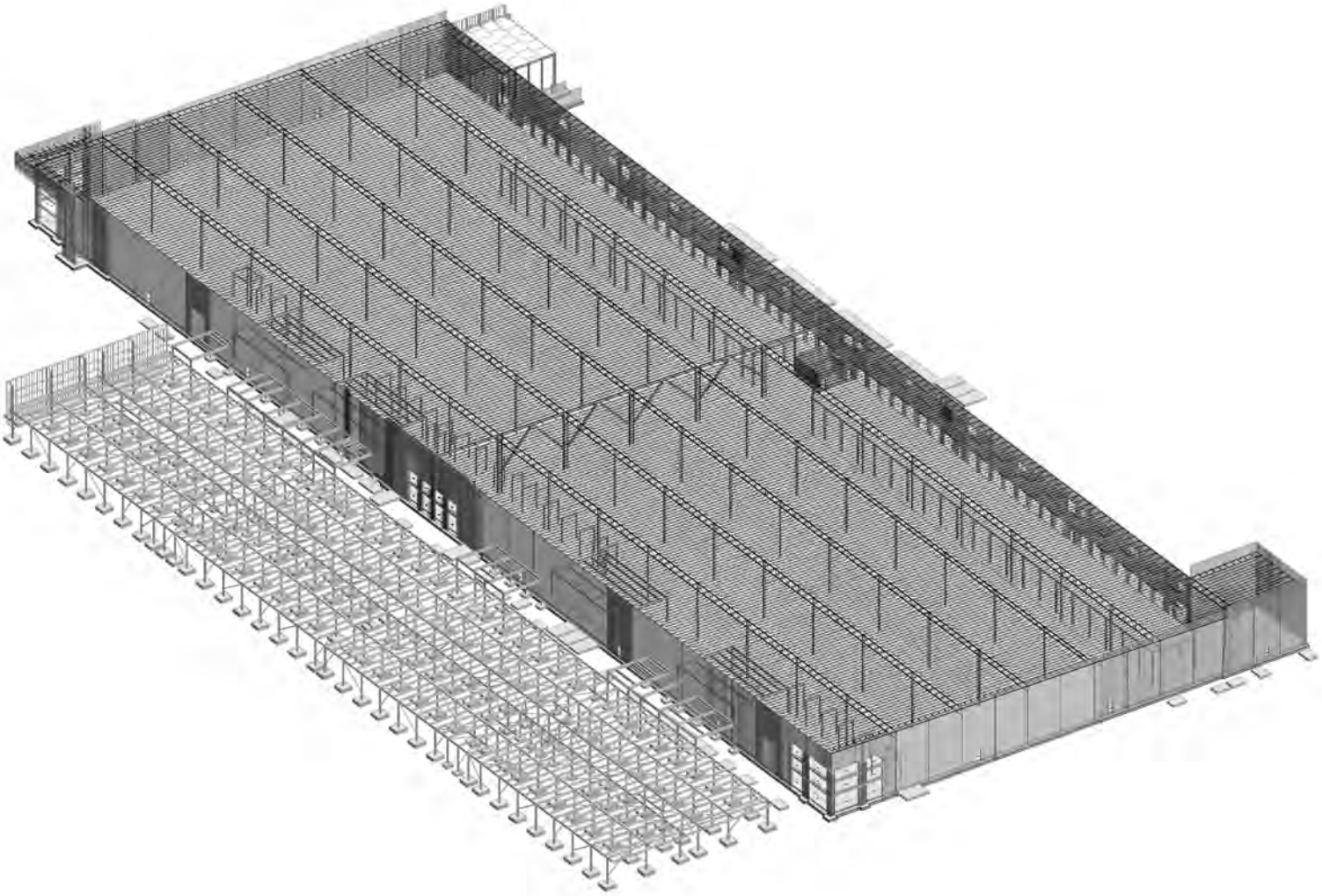
One Way Shear

Load Combination...	Left of Column #1			Right of Column #1			Left of Column #2			Right of Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	0.0	42.6	0.000	2.3	42.6	0.055	0.0	42.6	0.000	2.6	42.6	0.061
+1.20D	0.0	42.6	0.000	2.0	42.6	0.047	0.0	42.6	0.000	2.2	42.6	0.052
+1.20D+0.50W	0.0	42.6	0.000	5.9	42.6	0.138	0.0	42.6	0.000	2.6	42.6	0.060
+1.20D+W	0.0	42.6	0.000	6.4	42.6	0.149	0.0	42.6	0.000	9.9	42.6	0.232
+0.90D+W	0.0	42.6	0.000	4.8	42.6	0.112	0.0	42.6	0.000	8.4	42.6	0.197
+0.90D	0.0	42.6	0.000	1.5	42.6	0.035	0.0	42.6	0.000	1.7	42.6	0.039

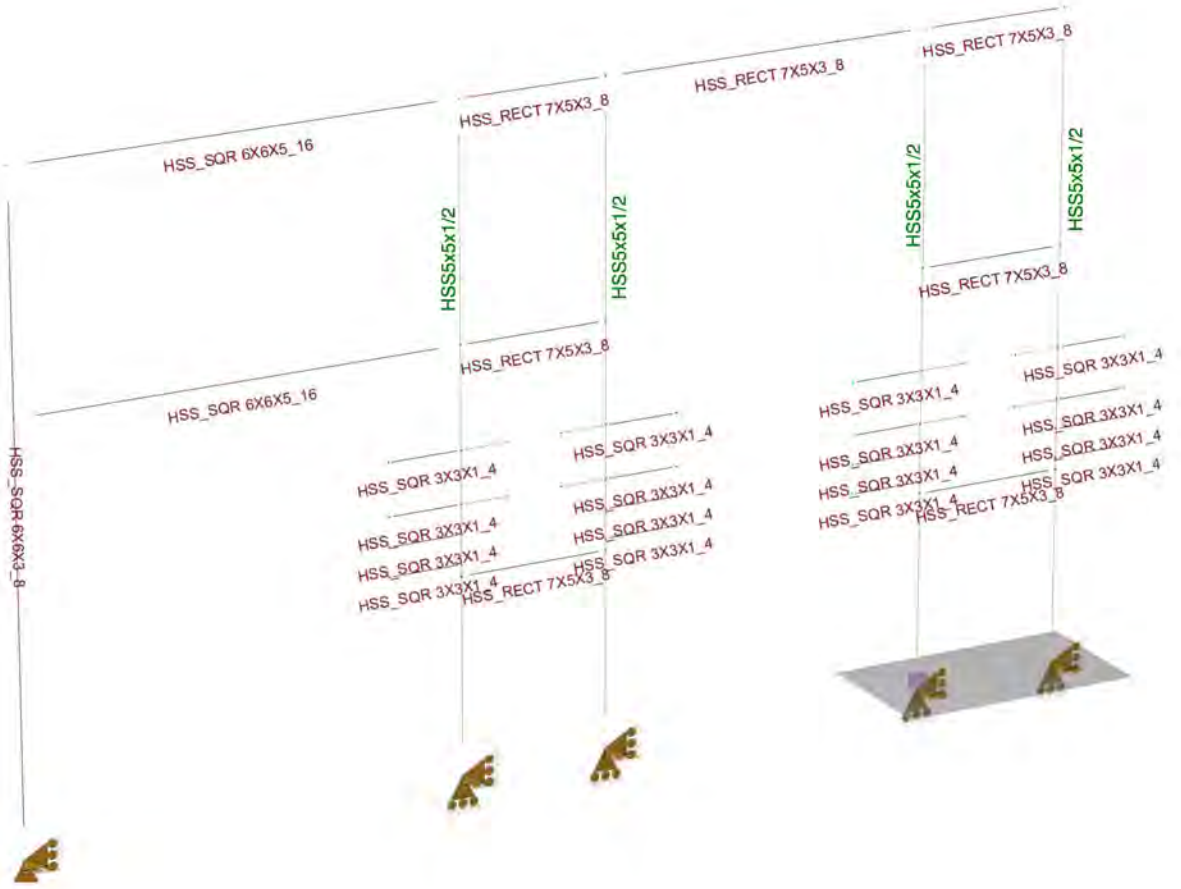
Two Way Shear

Load Combination...	Column #1			Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	0.8	164.3	0.005	1.7	164.3	0.010
+1.20D	0.7	164.3	0.004	1.4	164.3	0.009
+1.20D+0.50W	1.4	164.3	0.008	2.4	164.3	0.015
+1.20D+W	1.4	164.3	0.008	4.2	164.3	0.025
+0.90D+W	1.0	164.3	0.006	3.7	164.3	0.022
+0.90D	0.5	164.3	0.003	1.1	164.3	0.007

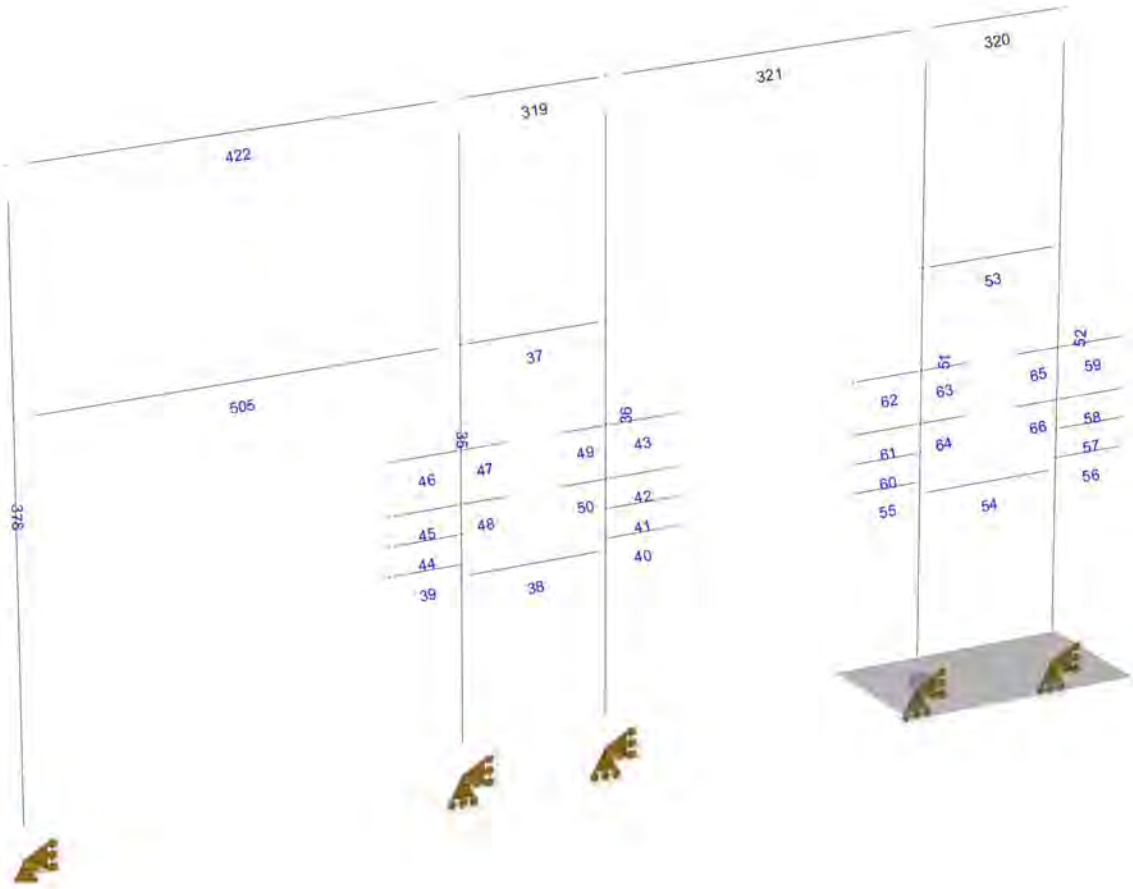
Chapter 4 - MEP Equipment Support Frames between Grids B/F and 3/17



MEP Equipment Support Frames between Grids B/F and 3/17 Member Sizes and Supports

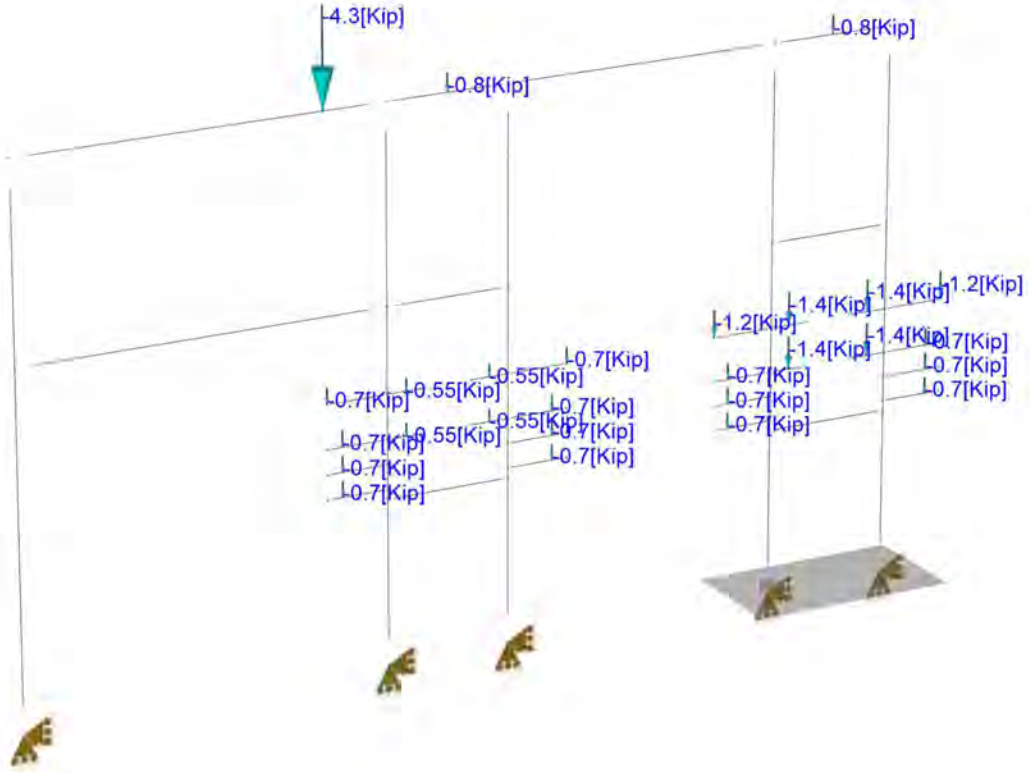


MEP Equipment Support Frames between Grids B/F and 3/17
Member Numbers



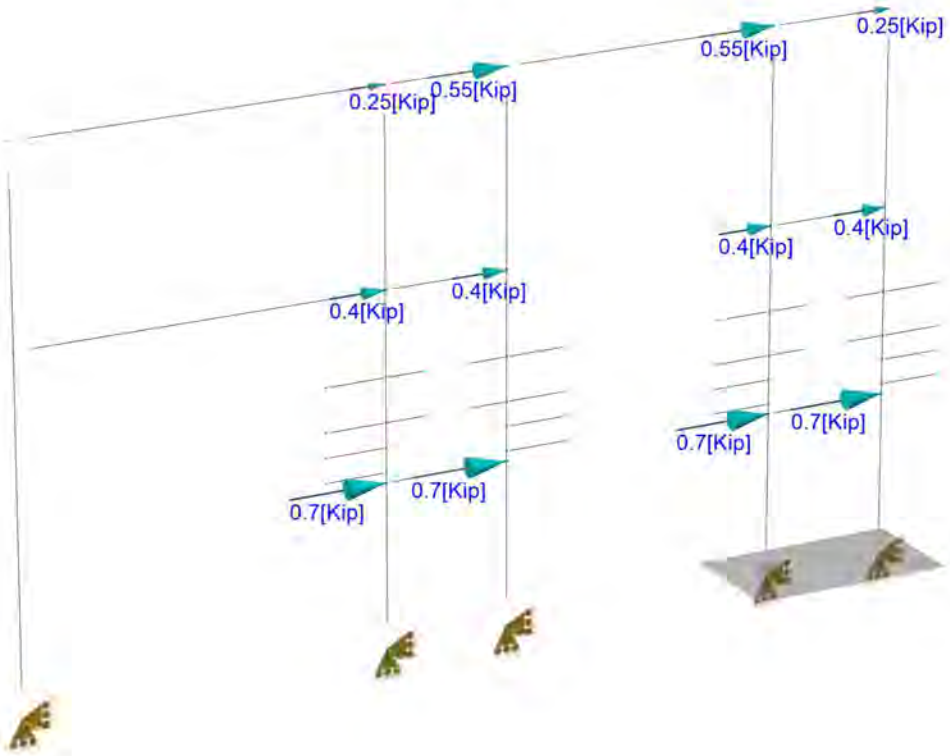
Loads
Concentrated user loads - Members

MEP Equipment Support Frames between Grids B/F and 3/17 Equipment Loads

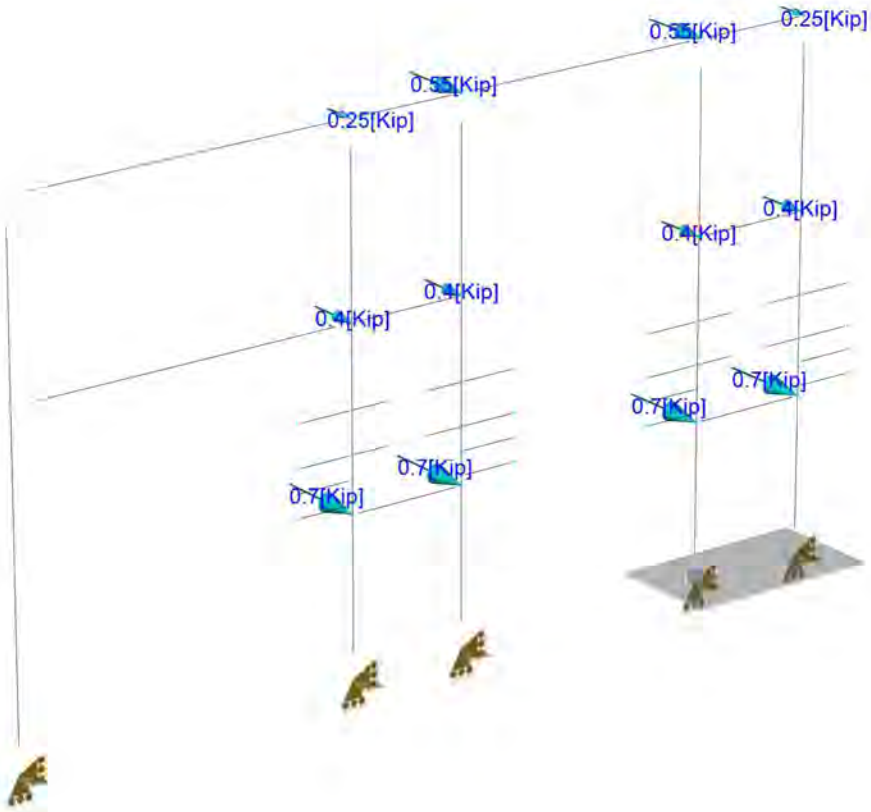


Loads
Concentrated user loads - Members

MEP Equipment Support Frames between Grids B/F and 3/17
Lateral Loads in X Direction



MEP Equipment Support Frames between Grids B/F and 3/17
Lateral Loads in Z Direction



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 11:41 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\HAC Framing\HAC Framing_REV 3 with Raised Upper Framing.rex

Steel Code Check
Summary - Group by description

Load conditions to be included in design :

D1=1.4DL1+1.4DL2
D2=1.2DL1+1.2DL2+E1
D3=1.2DL1+1.2DL2+E2
D4=0.9DL1+0.9DL2+E1
D5=0.9DL1+0.9DL2+E2

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Left Interior Cantilev...	*HSS_SQR 2X2X1_4*	63	D1 at 0.00%	0.55	OK	
Left Outside Cantilev...	*HSS_SQR 3X3X1_4*	62	D1 at 0.00%	0.58	OK	
Rack Column	*HSS_SQR 5X5X1_2*	51	D3 at 30.06%	0.47	OK	
Rack Long Upper Hei...	*HSS_RECT 7X5X3_8*	321	D1 at 100.00%	0.36	OK	
Rack Short Lower Be...		54	D2 at 100.00%	0.27	OK	
Rack Short Mid Heigh...		53	D2 at 0.00%	0.13	OK	
Rack Short Upper Hei...		320	D1 at 0.00%	0.27	OK	
Right Interior Cantile...	*HSS_SQR 2X2X1_4*	65	D1 at 0.00%	0.55	OK	
Right Outside Cantile...	*HSS_SQR 3X3X1_4*	59	D1 at 0.00%	**0.58**	**OK**	
Water Pipe Support B...	*HSS_SQR 6X6X5_16*	422	D1 at 100.00%	0.23	OK	
Water Pipe Support C...	*HSS_SQR 6X6X3_8*	378	D2 at 64.29%	0.07	OK	

Current Date: 8/17/2026 11:43 AM
Units system: English
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Steel Code Check
Comprehensive Report

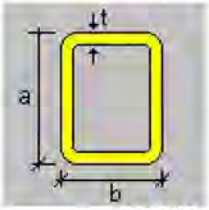
AISC 360-2016 LRFD (Hot-rolled)

Member : 63 (Left Interior Cantilever Beam) - OK

Section information

Section name: HSS_SQR 2X2X1_4 **(US)**

Dimensions



a	=	2.000	[in]	Height
b	=	2.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	1.510	
Moment of Inertia (local axes) (I)	[in4]	0.747	0.747
Moment of Inertia (principal axes) (I')	[in4]	0.747	0.747
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	0.703	0.703
Radius of gyration (principal axes) (r')	[in]	0.703	0.703
Saint-Venant torsion constant. (J)	[in4]	1.310	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	0.747	0.747
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	0.747	0.747
Plastic section modulus (local axis) (Z)	[in3]	0.964	0.964
Plastic section modulus (principal axis) (Z')	[in3]	0.964	0.964
Polar radius of gyration. (ro)	[in]	0.990	
Area for shear (Aw)	[in2]	0.606	0.606
Torsional constant. (C)	[in3]	1.405	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	2.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
2.00	2.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
2.00	2.00	2.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	67.95 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	67.95	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	75.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	62.40 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	62.40	Cl.E3
Effective length (KL_{33})	[ft]	2.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	34.12	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	245.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	45.92	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	69.34	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	62.40 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	62.40	Cl.E3
Effective length (KL_{22})	[ft]	2.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	34.12	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	245.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	45.92	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	69.34	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.55			
Capacity	:	3.62 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-1.97 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	3.62 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	16.37 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.12		
Capacity	:	16.37 [Kip]	Reference	: Cl.G1
Demand	:	1.97 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	3.16 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	3.16	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	3.51	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.55			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	Eq.H1-1b

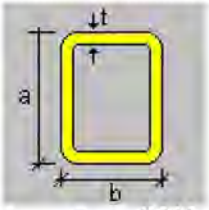
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.55	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	3.62	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	3.62	Cl.H1.1
Available axial strength (P_c)	[Kip]	67.95	Cl.H1.1

Member : 62 (Left Outside Cantilever Beam) - OK

Section information

Section name: HSS_SQR 3X3X1_4 **(US)**

Dimensions



a	=	3.000	[in]	Height
b	=	3.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	2.440	
Moment of Inertia (local axes) (I)	[in4]	3.020	3.020
Moment of Inertia (principal axes) (I')	[in4]	3.020	3.020
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.113	1.113
Radius of gyration (principal axes) (r')	[in]	1.113	1.113
Saint-Venant torsion constant. (J)	[in4]	5.080	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000

Top elastic section modulus of the section (local axis) (Ssup)	[in3]	2.010	2.010
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	2.010	2.010
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	2.010	2.010
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	2.010	2.010
Plastic section modulus (local axis) (Z)	[in3]	2.480	2.480
Plastic section modulus (principal axis) (Z')	[in3]	2.480	2.480
Polar radius of gyration. (ro)	[in]	1.571	
Area for shear (Aw)	[in2]	1.072	1.072
Torsional constant. (C)	[in3]	3.518	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	3.17

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
3.17	3.17

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
3.17	3.17	3.17	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00		
Capacity	:	109.80 [Kip]	Reference	: Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	109.80	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	122.00	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 100.82 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.88	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	9.88	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	100.82	Cl.E3
Effective length (KL_{33})	[ft]	3.17	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	34.16	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	245.34	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	2.44	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	45.91	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	112.02	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.00			
Capacity	: 100.82 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.88	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	9.88	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	100.82	Cl.E3
Effective length (KL_{22})	[ft]	3.17	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	34.16	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	245.34	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	2.44	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	45.91	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	112.02	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.58			
Capacity	:	9.30 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-5.38 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	9.30	Cl.F7.1
Yielding (M_n)	[Kip*ft]	10.33	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	9.30 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	9.30	Cl.F7.1
Yielding (M_n)	[Kip*ft]	10.33	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	28.95 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	28.95	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	32.17	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.06			
Capacity	:	28.95 [Kip]	Reference	:	Cl.G1
Demand	:	1.72 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	28.95	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	32.17	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	7.91 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	7.91	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	8.79	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.58			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	Eq.H1-1b

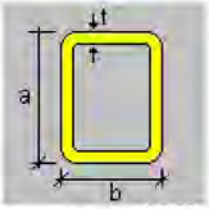
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.58	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	9.30	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	9.30	Cl.H1.1
Available axial strength (P_c)	[Kip]	109.80	Cl.H1.1

Member : 51 (Rack Column) - OK

Section information

Section name: HSS_SQR 5X5X1_2 **(US)**

Dimensions



a	=	5.000	[in]	Height
b	=	5.000	[in]	Width
T	=	0.465	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.880	
Moment of Inertia (local axes) (I)	[in4]	26.000	26.000
Moment of Inertia (principal axes) (I')	[in4]	26.000	26.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.816	1.816
Radius of gyration (principal axes) (r')	[in]	1.816	1.816
Saint-Venant torsion constant. (J)	[in4]	44.600	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	10.400	10.400
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	10.400	10.400
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	10.400	10.400
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	10.400	10.400
Plastic section modulus (local axis) (Z)	[in3]	13.100	13.100
Plastic section modulus (principal axis) (Z')	[in3]	13.100	13.100
Polar radius of gyration. (ro)	[in]	2.562	
Area for shear (Aw)	[in2]	3.353	3.353
Torsional constant. (C)	[in3]	18.729	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	28.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
28.00	28.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
18.00	18.00	28.00	1.0	1.0	1.0
7.00	7.00		1.0	1.0	
3.00	3.00		1.0	1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 354.60 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	354.60	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	394.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.25			
Capacity	: 125.89 [Kip]	Reference	: Cl.E3	
Demand	: 31.33 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	7.75	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	7.75	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	125.89	Cl.E3
Effective length (KL_{33})	[ft]	18.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	118.91	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	20.24	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.88	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	17.75	Eq.E3-3
Nominal flexural buckling strength (P_{n33})	[Kip]	139.88	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.25			
Capacity	:	125.89 [Kip]	Reference	:	Cl.E3
Demand	:	31.33 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.75	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	7.75	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	125.89	Cl.E3
Effective length (KL_{22})	[ft]	18.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	118.91	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	20.24	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.88	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	17.75	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	139.88	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.27			
Capacity	:	49.13 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	13.09 [Kip*ft]	Ctrl Eq.	:	D2 at 30.06%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.75	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	7.75	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	49.13	Cl.F7.1
Yielding (M_n)	[Kip*ft]	54.58	Eq.F7-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	49.13	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	16.33	Eq.F7-12
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	452.17	Eq.F7-13
Lateral-torsional buckling modification factor (C_b)	--	2.35	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	54.58	Eq.F7-10

Bending about minor axis, M22

Ratio	:	0.26			
Capacity	:	49.13 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-12.85 [Kip*ft]	Ctrl Eq.	:	D3 at 30.06%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.75	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	7.75	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	49.13	Cl.F7.1
Yielding (M_n)	[Kip*ft]	54.58	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.01			
Capacity	:	90.52 [Kip]	Reference	:	Cl.G1
Demand	:	1.31 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	90.52	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	100.58	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.02			
Capacity	:	90.52 [Kip]	Reference	:	Cl.G1
Demand	:	1.38 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	90.52	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	100.58	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	42.14 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.04 [Kip*ft]	Ctrl Eq.	:	D2 at 64.29%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	42.14	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	46.82	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.47			
Ctrl Eq.	:	D3 at 30.06%	Reference	:	Eq.H1-1a

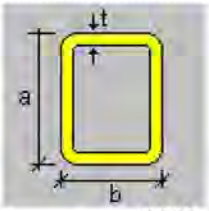
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.47	Eq.H1-1a
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	49.13	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	49.13	Cl.H1.1
Available axial strength (P_c)	[Kip]	125.89	Cl.H1.1

Member : 321 (Rack Long Upper Height Beam) - OK

Section information

Section name: HSS_RECT 7X5X3_8 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	5.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	49.500	29.300
Moment of Inertia (principal axes) (I')	[in4]	49.500	29.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.555	1.966
Radius of gyration (principal axes) (r')	[in]	2.555	1.966
Saint-Venant torsion constant. (J)	[in4]	60.600	
Section warping constant. (Cw)	[in6]	4.982	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.100	11.700
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.100	11.700
Plastic section modulus (local axis) (Z)	[in3]	17.500	13.800
Plastic section modulus (principal axis) (Z')	[in3]	17.500	13.800
Polar radius of gyration. (ro)	[in]	3.222	
Area for shear (Aw)	[in2]	2.759	4.155
Torsional constant. (C)	[in3]	21.424	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	14.42

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
14.42	14.42

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
14.42	14.42	14.42	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.10 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.10	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	243.98 [Kip]	Reference	:	Cl.E3
Demand	:	0.29 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	11.33	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.06	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	243.98	Cl.E3
Effective length (KL_{33})	[ft]	14.42	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	67.70	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	62.45	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	35.76	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	271.09	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	193.65 [Kip]	Reference	:	Cl.E3
Demand	:	0.29 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	11.33	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.06	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	193.65	Cl.E3
Effective length (KL_{22})	[ft]	14.42	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	87.99	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	36.97	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	28.39	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	215.17	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.36			
Capacity	:	65.63 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-23.42 [Kip*ft]	Ctrl Eq.	:	D1 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.63	Cl.F7.1
Yielding (M_n)	[Kip*ft]	72.92	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.02		
Capacity	:	51.75 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-0.80 [Kip*ft]	Ctrl Eq.	: D2 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.75	Cl.F7.1
Yielding (M_n)	[Kip*ft]	57.50	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	74.50 [Kip]	Reference	: Cl.G1
Demand	:	0.21 [Kip]	Ctrl Eq.	: D2 at 71.68%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	74.50	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	82.78	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.09		
Capacity	:	112.19 [Kip]	Reference	: Cl.G1
Demand	:	10.16 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	112.19	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	124.66	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	48.20 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.01 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	48.20	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	53.56	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.36			
Ctrl Eq.	:	D1 at 100.00%	Reference	:	Eq.H1-1b

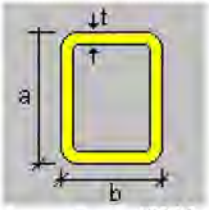
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.36	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	65.63	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.75	Cl.H1.1
Available axial strength (P_c)	[Kip]	193.65	Cl.H1.1

Member : 54 (Rack Short Lower Beam) - OK

Section information

Section name: HSS_RECT 7X5X3_8 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	5.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	49.500	29.300
Moment of Inertia (principal axes) (I')	[in4]	49.500	29.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.555	1.966
Radius of gyration (principal axes) (r')	[in]	2.555	1.966
Saint-Venant torsion constant. (J)	[in4]	60.600	
Section warping constant. (Cw)	[in6]	4.982	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000

Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.100	11.700
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.100	11.700
Plastic section modulus (local axis) (Z)	[in3]	17.500	13.800
Plastic section modulus (principal axis) (Z')	[in3]	17.500	13.800
Polar radius of gyration. (ro)	[in]	3.222	
Area for shear (Aw)	[in2]	2.759	4.155
Torsional constant. (C)	[in3]	21.424	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	6.42

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
6.42	6.42

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
6.42	6.42	6.42	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.10 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.10	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.00	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	319.19 [Kip]	Reference	:	Cl.E3
Demand	:	1.73 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	11.33	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	17.06	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	319.19	Cl.E3
Effective length (KL_{33})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	30.13	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	315.24	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	46.79	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	354.66	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.01			
Capacity	:	304.91 [Kip]	Reference	:	Cl.E3
Demand	:	1.73 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	11.33	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	17.06	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	304.91	Cl.E3
Effective length (KL_{22})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	39.16	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	186.60	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	44.70	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	338.79	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.25			
Capacity	:	65.63 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-16.46 [Kip*ft]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.63	Cl.F7.1
Yielding (M_n)	[Kip*ft]	72.92	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.02			
Capacity	:	51.75 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	1.24 [Kip*ft]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.75	Cl.F7.1
Yielding (M_n)	[Kip*ft]	57.50	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	74.50 [Kip]	Reference	:	Cl.G1
Demand	:	0.36 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	74.50	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	82.78	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.04			
Capacity	:	112.19 [Kip]	Reference	:	Cl.G1
Demand	:	4.92 [Kip]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	112.19	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	124.66	Eq.G4-1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	48.20 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.02 [Kip*ft]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	48.20	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	53.56	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.27			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Eq.H1-1b

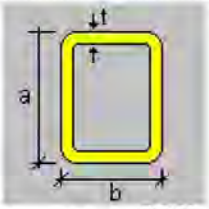
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.27	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	65.63	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.75	Cl.H1.1
Available axial strength (P_c)	[Kip]	304.91	Cl.H1.1

Member : 53 (Rack Short Mid Height Beam) - OK

Section information

Section name: HSS_RECT 7X5X3_8 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	5.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	49.500	29.300
Moment of Inertia (principal axes) (I')	[in4]	49.500	29.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.555	1.966
Radius of gyration (principal axes) (r')	[in]	2.555	1.966
Saint-Venant torsion constant. (J)	[in4]	60.600	
Section warping constant. (Cw)	[in6]	4.982	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.100	11.700
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.100	11.700
Plastic section modulus (local axis) (Z)	[in3]	17.500	13.800
Plastic section modulus (principal axis) (Z')	[in3]	17.500	13.800
Polar radius of gyration. (ro)	[in]	3.222	
Area for shear (Aw)	[in2]	2.759	4.155
Torsional constant. (C)	[in3]	21.424	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	6.42

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
6.42	6.42

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
6.42	6.42	6.42	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 341.10 [Kip]	Reference	: Cl.D2	
Demand	: 1.30 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.10	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 319.19 [Kip]	Reference	: Cl.E3	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	11.33	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.06	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	319.19	Cl.E3
Effective length (KL_{33})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	30.13	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	315.24	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	46.79	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	354.66	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	304.91 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	11.33	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.06	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	304.91	Cl.E3
Effective length (KL_{22})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	39.16	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	186.60	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	44.70	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	338.79	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.11			
Capacity	:	65.63 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	6.93 [Kip*ft]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.63	Cl.F7.1
Yielding (M_n)	[Kip*ft]	72.92	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.03			
Capacity	:	51.75 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-1.45 [Kip*ft]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	

Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.75	Cl.F7.1
Yielding (M_n)	[Kip*ft]	57.50	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.01		
Capacity	:	74.50 [Kip]	Reference	: Cl.G1
Demand	:	0.42 [Kip]	Ctrl Eq.	: D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	74.50	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	82.78	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.02		
Capacity	:	112.19 [Kip]	Reference	: Cl.G1
Demand	:	1.81 [Kip]	Ctrl Eq.	: D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	112.19	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	124.66	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00		
Capacity	:	48.20 [Kip*ft]	Reference	: Cl.H3.1
Demand	:	-0.01 [Kip*ft]	Ctrl Eq.	: D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	48.20	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	53.56	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.13		
Ctrl Eq.	:	D2 at 0.00%	Reference	: Eq.H1-1b

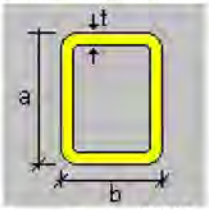
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.13	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	65.63	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.75	Cl.H1.1
Available axial strength (P_c)	[Kip]	341.10	Cl.H1.1

Member : 320 (Rack Short Upper Height Beam) - OK

Section information

Section name: HSS_RECT 7X5X3_8 **(US)**

Dimensions



a	=	7.000	[in]	Height
b	=	5.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	49.500	29.300
Moment of Inertia (principal axes) (I')	[in4]	49.500	29.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.555	1.966
Radius of gyration (principal axes) (r')	[in]	2.555	1.966
Saint-Venant torsion constant. (J)	[in4]	60.600	
Section warping constant. (Cw)	[in6]	4.982	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	14.100	11.700
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	14.100	11.700
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	14.100	11.700
Plastic section modulus (local axis) (Z)	[in3]	17.500	13.800
Plastic section modulus (principal axis) (Z')	[in3]	17.500	13.800
Polar radius of gyration. (ro)	[in]	3.222	
Area for shear (Aw)	[in2]	2.759	4.155
Torsional constant. (C)	[in3]	21.424	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	6.42

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
6.42	6.42

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
6.42	6.42	6.42	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.10 [Kip]	Reference	:	Cl.D2
Demand	:	0.33 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.10	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	319.19 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	11.33	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	

Stiffened element slenderness (λ)	–	17.06	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	319.19	Cl.E3
Effective length (KL_{33})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	30.13	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	315.24	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	46.79	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	354.66	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00		
Capacity	:	304.91 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	11.33	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	17.06	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	304.91	Cl.E3
Effective length (KL_{22})	[ft]	6.42	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	39.16	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	186.60	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	44.70	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	338.79	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.27		
Capacity	:	65.63 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-17.43 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	65.63	Cl.F7.1
Yielding (M_n)	[Kip*ft]	72.92	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.02			
Capacity	:	51.75 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-1.17 [Kip*ft]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	11.33	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.06	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.75	Cl.F7.1
Yielding (M_n)	[Kip*ft]	57.50	Eq.F7-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	74.50 [Kip]	Reference	:	Cl.G1
Demand	:	0.33 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	74.50	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	82.78	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.03			
Capacity	:	112.19 [Kip]	Reference	:	Cl.G1
Demand	:	3.78 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	112.19	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	124.66	Eq.G4-1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	48.20 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.04 [Kip*ft]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	48.20	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	53.56	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio : 0.27
Ctrl Eq. : D1 at 0.00% Reference : Eq.H1-1b

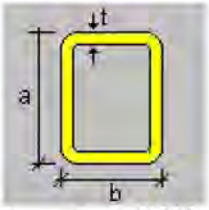
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.27	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	65.63	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.75	Cl.H1.1
Available axial strength (P_c)	[Kip]	341.10	Cl.H1.1

Member : 65 (Right Interior Cantilever Beam) - OK

Section information

Section name: HSS_SQR 2X2X1_4 **(US)**

Dimensions



a	=	2.000	[in]	Height
b	=	2.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	1.510	
Moment of Inertia (local axes) (I)	[in4]	0.747	0.747
Moment of Inertia (principal axes) (I')	[in4]	0.747	0.747
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	0.703	0.703
Radius of gyration (principal axes) (r')	[in]	0.703	0.703
Saint-Venant torsion constant. (J)	[in4]	1.310	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	0.747	0.747
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	0.747	0.747
Plastic section modulus (local axis) (Z)	[in3]	0.964	0.964
Plastic section modulus (principal axis) (Z')	[in3]	0.964	0.964

Polar radius of gyration. (ro)	[in]	0.990	
Area for shear (Aw)	[in2]	0.606	0.606
Torsional constant. (C)	[in3]	1.405	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	2.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
2.00	2.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
2.00	2.00	2.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	67.95 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	67.95	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	75.50	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	62.40 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	62.40	Cl.E3
Effective length (KL_{33})	[ft]	2.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	34.12	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	245.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	45.92	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	69.34	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	62.40 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	62.40	Cl.E3
Effective length (KL_{22})	[ft]	2.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	34.12	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	245.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	45.92	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	69.34	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.55			
Capacity	:	3.62 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-1.97 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	3.62 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	16.37 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.12			
Capacity	:	16.37 [Kip]	Reference	:	Cl.G1
Demand	:	1.97 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	3.16 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	3.16	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	3.51	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.55			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	Eq.H1-1b

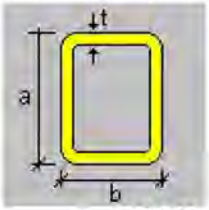
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.55	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	3.62	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	3.62	Cl.H1.1
Available axial strength (P_c)	[Kip]	67.95	Cl.H1.1

Member : 59 (Right Outside Cantilever Beam) - OK

Section information

Section name: HSS_SQR 3X3X1_4 **(US)**

Dimensions



a	=	3.000	[in]	Height
b	=	3.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	2.440	
Moment of Inertia (local axes) (I)	[in4]	3.020	3.020
Moment of Inertia (principal axes) (I')	[in4]	3.020	3.020
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.113	1.113
Radius of gyration (principal axes) (r')	[in]	1.113	1.113
Saint-Venant torsion constant. (J)	[in4]	5.080	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	2.010	2.010
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	2.010	2.010
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	2.010	2.010
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	2.010	2.010
Plastic section modulus (local axis) (Z)	[in3]	2.480	2.480
Plastic section modulus (principal axis) (Z')	[in3]	2.480	2.480
Polar radius of gyration. (ro)	[in]	1.571	
Area for shear (Aw)	[in2]	1.072	1.072
Torsional constant. (C)	[in3]	3.518	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	3.17

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
3.17	3.17

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
3.17	3.17	3.17	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design

Axial tension

Ratio	:	0.00		
Capacity	:	109.80 [Kip]	Reference	: Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	109.80	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	122.00	Eq.D2-1

Axial Compression Design

Compression in the major axis 33

Ratio	:	0.00		
Capacity	:	100.82 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.88	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	9.88	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	100.82	Cl.E3
Effective length (KL_{33})	[ft]	3.17	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	34.16	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	245.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	2.44	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	45.91	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	112.02	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00		
Capacity	:	100.82 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	9.88	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	9.88	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	100.82	Cl.E3
Effective length (KL_{22})	[ft]	3.17	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	34.16	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	245.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	2.44	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	45.91	Eq.E3-2

Nominal flexural buckling strength (P_{n22})	[Kip]	112.02	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.58		
Capacity	:	9.30 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-5.38 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	9.30	Cl.F7.1
Yielding (M_n)	[Kip*ft]	10.33	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	9.30 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	9.88	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	9.30	Cl.F7.1
Yielding (M_n)	[Kip*ft]	10.33	Eq.F7-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	28.95 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	28.95	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	32.17	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.06			
Capacity	:	28.95 [Kip]	Reference	:	Cl.G1
Demand	:	1.72 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	28.95	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	32.17	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	7.91 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	7.91	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	8.79	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.58			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	Eq.H1-1b

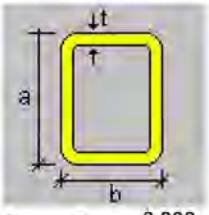
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.58	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	9.30	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	9.30	Cl.H1.1
Available axial strength (P_c)	[Kip]	109.80	Cl.H1.1

Member : 422 (Water Pipe Support Beam X Dir) - OK

Section information

Section name: HSS_SQR 6X6X5_16 **(US)**

Dimensions



a	=	6.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.291	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	6.430	
Moment of Inertia (local axes) (I)	[in4]	34.300	34.300
Moment of Inertia (principal axes) (I')	[in4]	34.300	34.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.310	2.310
Radius of gyration (principal axes) (r')	[in]	2.310	2.310
Saint-Venant torsion constant. (J)	[in4]	55.400	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	11.400	11.400
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	11.400	11.400
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	11.400	11.400
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	11.400	11.400
Plastic section modulus (local axis) (Z)	[in3]	13.600	13.600
Plastic section modulus (principal axis) (Z')	[in3]	13.600	13.600
Polar radius of gyration. (ro)	[in]	3.266	
Area for shear (Aw)	[in2]	2.984	2.984
Torsional constant. (C)	[in3]	18.871	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	19.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
19.00	19.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
19.00	19.00	19.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	289.35 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	289.35	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	321.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	141.90 [Kip]	Reference	:	Cl.E3
Demand	:	0.36 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	17.62	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.62	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	141.90	Cl.E3
Effective length (KL_{33})	[ft]	19.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	98.72	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	29.37	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.43	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	24.52	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	157.66	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	141.90 [Kip]	Reference	:	Cl.E3
Demand	:	0.36 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	17.62	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	17.62	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	141.90	Cl.E3
Effective length (KL_{22})	[ft]	19.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	98.72	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	29.37	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.43	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	24.52	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	157.66	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.23			
Capacity	:	51.00 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-11.83 [Kip*ft]	Ctrl Eq.	:	D1 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	17.62	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.62	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	51.00	Cl.F7.1
Yielding (M_n)	[Kip*ft]	56.67	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.01			
Capacity	:	51.00 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-0.39 [Kip*ft]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	17.62	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	17.62	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	

Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.00	Cl.F7.1
Yielding (M_n)	[Kip*ft]	56.67	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	80.57 [Kip]	Reference	: Cl.G1
Demand	:	0.02 [Kip]	Ctrl Eq.	: D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	80.57	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	89.52	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.07		
Capacity	:	80.57 [Kip]	Reference	: Cl.G1
Demand	:	5.73 [Kip]	Ctrl Eq.	: D1 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	80.57	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	89.52	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.01		
Capacity	:	42.46 [Kip*ft]	Reference	: Cl.H3.1
Demand	:	-0.48 [Kip*ft]	Ctrl Eq.	: D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	42.46	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	47.18	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.23		
Ctrl Eq.	:	D1 at 100.00%	Reference	: Eq.H1-1b

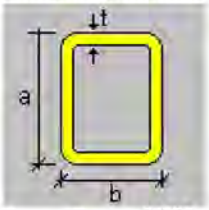
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.23	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	51.00	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.00	Cl.H1.1
Available axial strength (P_c)	[Kip]	141.90	Cl.H1.1

Member : 378 (Water Pipe Support Column) - OK

Section information

Section name: HSS_SQR 6X6X3_8 **(US)**

Dimensions



a	=	6.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	39.500	39.500
Moment of Inertia (principal axes) (I')	[in4]	39.500	39.500
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.283	2.283
Radius of gyration (principal axes) (r')	[in]	2.283	2.283
Saint-Venant torsion constant. (J)	[in4]	64.600	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	13.200	13.200
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	13.200	13.200
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	13.200	13.200
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	13.200	13.200
Plastic section modulus (local axis) (Z)	[in3]	15.800	15.800
Plastic section modulus (principal axis) (Z')	[in3]	15.800	15.800
Polar radius of gyration. (ro)	[in]	3.225	
Area for shear (Aw)	[in2]	3.457	3.457
Torsional constant. (C)	[in3]	22.122	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	28.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
28.00	28.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
18.00	18.00	28.00	1.0	1.0	1.0
7.00	7.00		1.0	1.0	
3.00	3.00		1.0	1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	341.10 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	341.10	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	379.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	177.25 [Kip]	Reference	:	Cl.E3
Demand	:	2.11 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	177.25	Cl.E3
Effective length (KL_{33})	[ft]	18.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	94.62	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	31.97	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	25.98	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	196.94	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.01			
Capacity	:	177.25 [Kip]	Reference	:	Cl.E3
Demand	:	2.11 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	177.25	Cl.E3
Effective length (KL_{22})	[ft]	18.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	94.62	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	31.97	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	25.98	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	196.94	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.07			
Capacity	:	59.25 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	3.93 [Kip*ft]	Ctrl Eq.	:	D2 at 64.29%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	

Factored yielding strength(ϕM_n):	[Kip*ft]	59.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	65.83	Eq.F7-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	59.25	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	20.09	Eq.F7-12
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	528.47	Eq.F7-13
Lateral-torsional buckling modification factor (C_b)	--	1.74	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	65.83	Eq.F7-10

Bending about minor axis, M22

Ratio	: 0.02			
Capacity	: 59.25 [Kip*ft]	Reference	: Cl.F7.1	
Demand	: -1.23 [Kip*ft]	Ctrl Eq.	: D3 at 64.29%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	59.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	65.83	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	: 0.00			
Capacity	: 93.34 [Kip]	Reference	: Cl.G1	
Demand	: 0.06 [Kip]	Ctrl Eq.	: D3 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	93.34	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	103.72	Eq.G4-1

Shear in minor axis 22

Ratio	: 0.00			
Capacity	: 93.34 [Kip]	Reference	: Cl.G1	
Demand	: 0.36 [Kip]	Ctrl Eq.	: D1 at 64.29%	

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	93.34	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	103.72	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.01			
Capacity	:	49.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.39 [Kip*ft]	Ctrl Eq.	:	D3 at 64.29%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	49.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	55.30	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.07			
Ctrl Eq.	:	D2 at 64.29%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.07	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	59.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	59.25	Cl.H1.1
Available axial strength (P_c)	[Kip]	177.25	Cl.H1.1

MEP Equipment Support Frames between Grids B/F and 3/17 Beam to Column Connection

Project data

Project name	Project Sunshine
Project number	
Author	PD
Description	Connection Design
Date	1/21/2026
Code	AISC/ACI

Material

Steel	A992, A572 Gr.50, A1085. Gr.A, A500. Gr. C, A36

Project item Project Sunshine

Name	Project Sunshine
Description	WIF Moment End Plate
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC 360-16 (LRFD) / ACI 318-19

Design

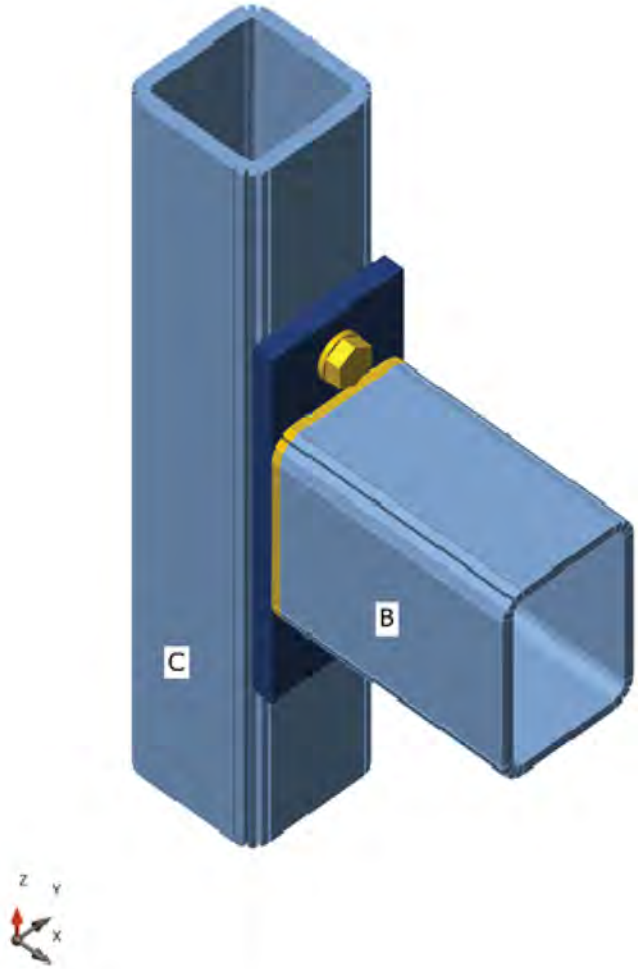
Members

Geometry

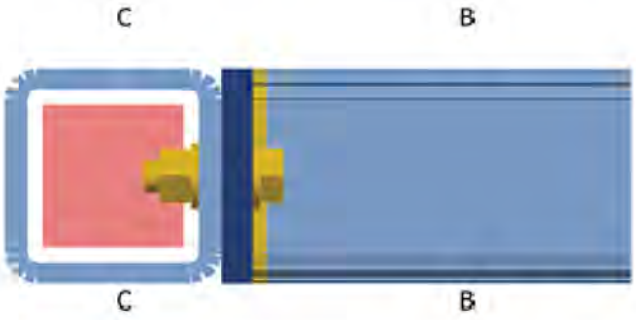
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
C	3 - RHS5x5(RHS127x127)	0.00	90.00	0.00	0.00	0.00	0.00
B	4 - RHS7x5(RHS177x127)	0.00	0.00	0.00	0.00	0.00	0.00

Supports and forces

Name	Support	Forces in	X [in]
C / begin	N-Vy-Vz-Mx-My-Mz	Node	0.00
C / end		Node	0.00
B / end		Node	0.00

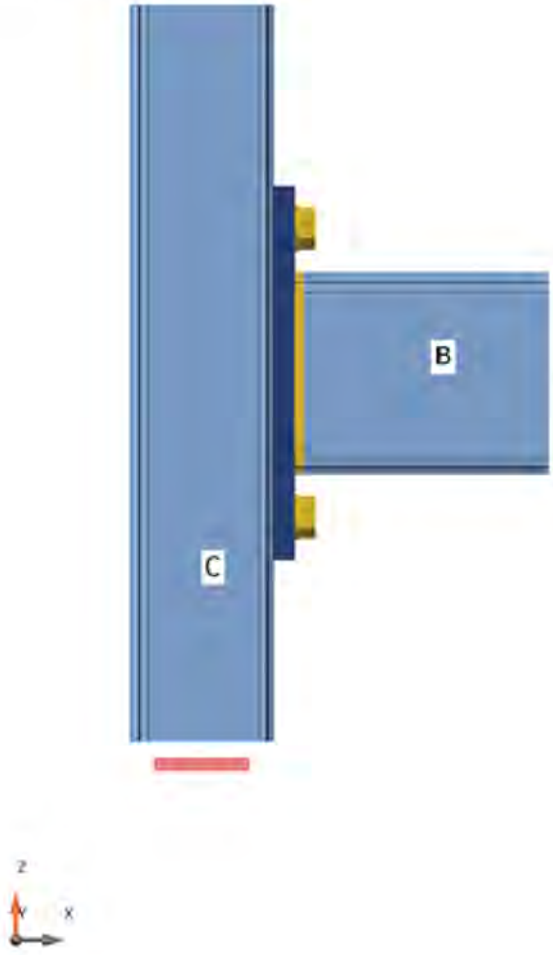


Project Sunshine, Axonometric view





Project Sunshine, Z direction view

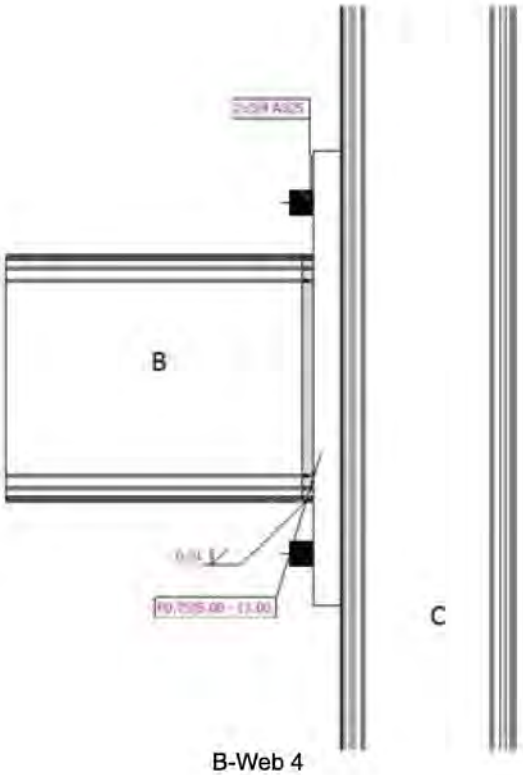


Project Sunshine, Y direction view





Project Sunshine, X direction view

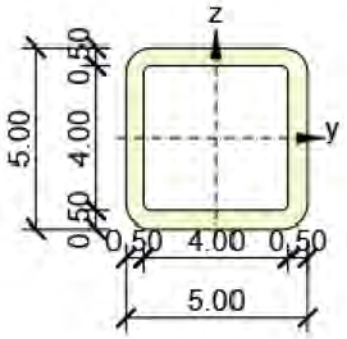
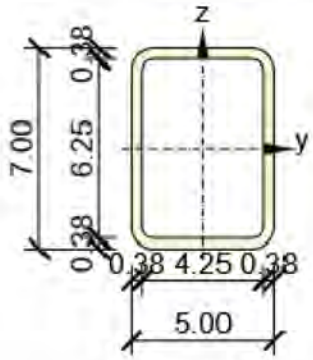


B-Web 4

Cross-sections

Name	Material
3 - RHS5x5(RHS127x127)	A500. Gr. C
4 - RHS7x5(RHS177x127)	A500. Gr. C

Cross-sections

Name	Material	Drawing
3 - RHS5x5(RHS127x127)	A500. Gr. C	
4 - RHS7x5(RHS177x127)	A500. Gr. C	

Bolts

Name	Diameter [in]	f_y [ksi]	f_u [ksi]	Gross area [in^2]
3/4 A325	0.75	92.00	120.00	0.44

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	C / Begin	0.00	0.00	0.00	0.00	0.00	0.00
	C / End	0.00	0.00	0.00	0.00	0.00	0.00
	B / End	2.00	0.00	-5.50	0.00	18.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	2.00	0.00	-5.50	0.00	18.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.63 < 5.00%	OK
Loc. deformation	0.4 < 3%	OK
Bolts	83.9 < 100%	OK
Welds	85.3 < 100%	OK
Buckling	68.96	
GMNA	Calculated	

Plates

Name	Material	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
C	A500. Gr. C	0.50	LE1	45.18	0.63	16.17	OK
B	A500. Gr. C	0.38	LE1	41.00	0.01	0.00	OK
EP1	A36	0.75	LE1	32.87	1.63	9.65	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A500. Gr. C	50.00	5.00
A36	36.00	5.00

Detailed result for C

Design values used in the analysis

$\phi F_y =$ 45.00 ksi

Where:

$F_y = 50.00$ ksi – characteristic yield strength

$\phi = 0.90$ – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for B

Design values used in the analysis

$\phi F_y =$ 45.00 ksi

Where:

$F_y = 50.00$ ksi – characteristic yield strength

$\phi = 0.90$ – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for EP1

Design values used in the analysis

$\phi F_y =$ 32.40 ksi

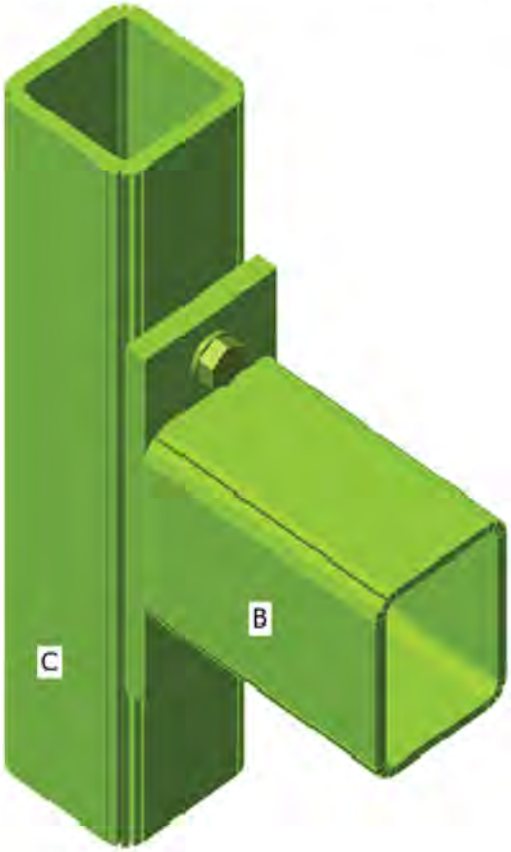
Where:

$F_y = 36.00$ ksi – characteristic yield strength

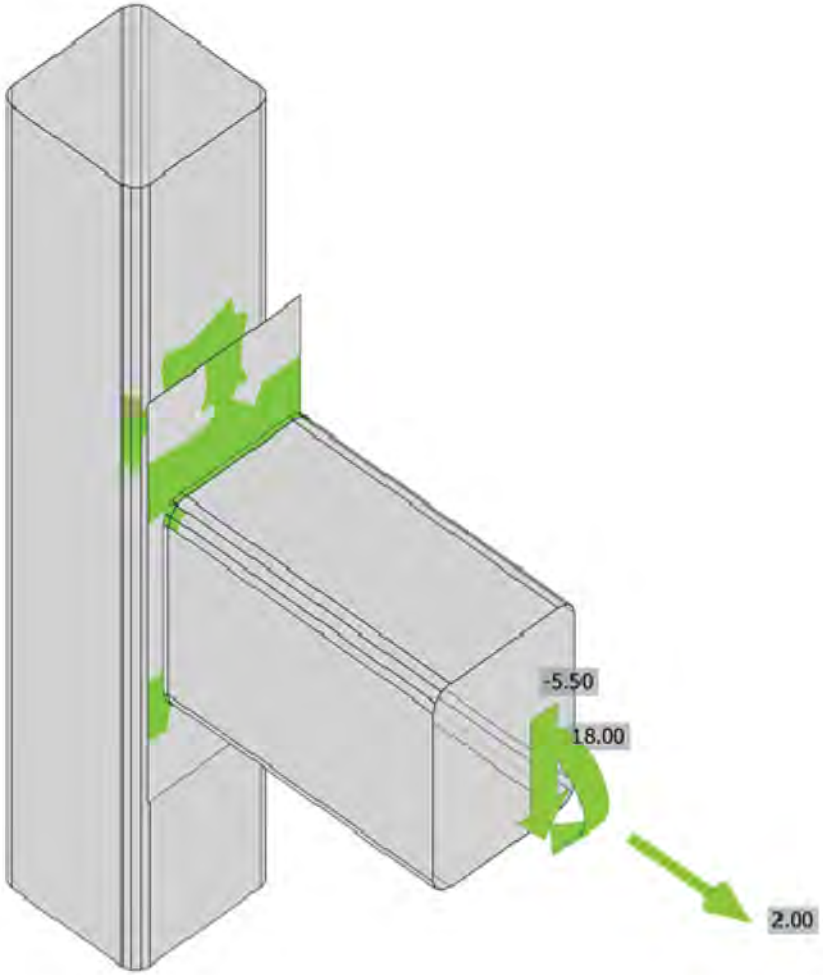
$\phi = 0.90$ – resistance factor for steel material AISC 360-16 – B3.1

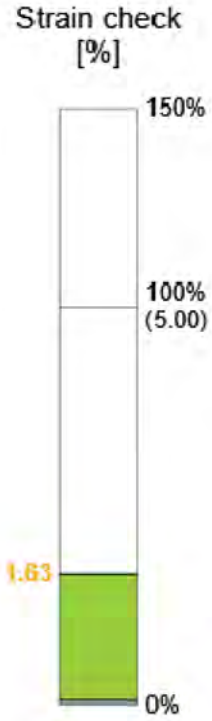
Loc. deformation

Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
C	5.00	LE1	0.02	0.15	0.40	OK
B	5.00	LE1	0.00	0.15	0.08	OK

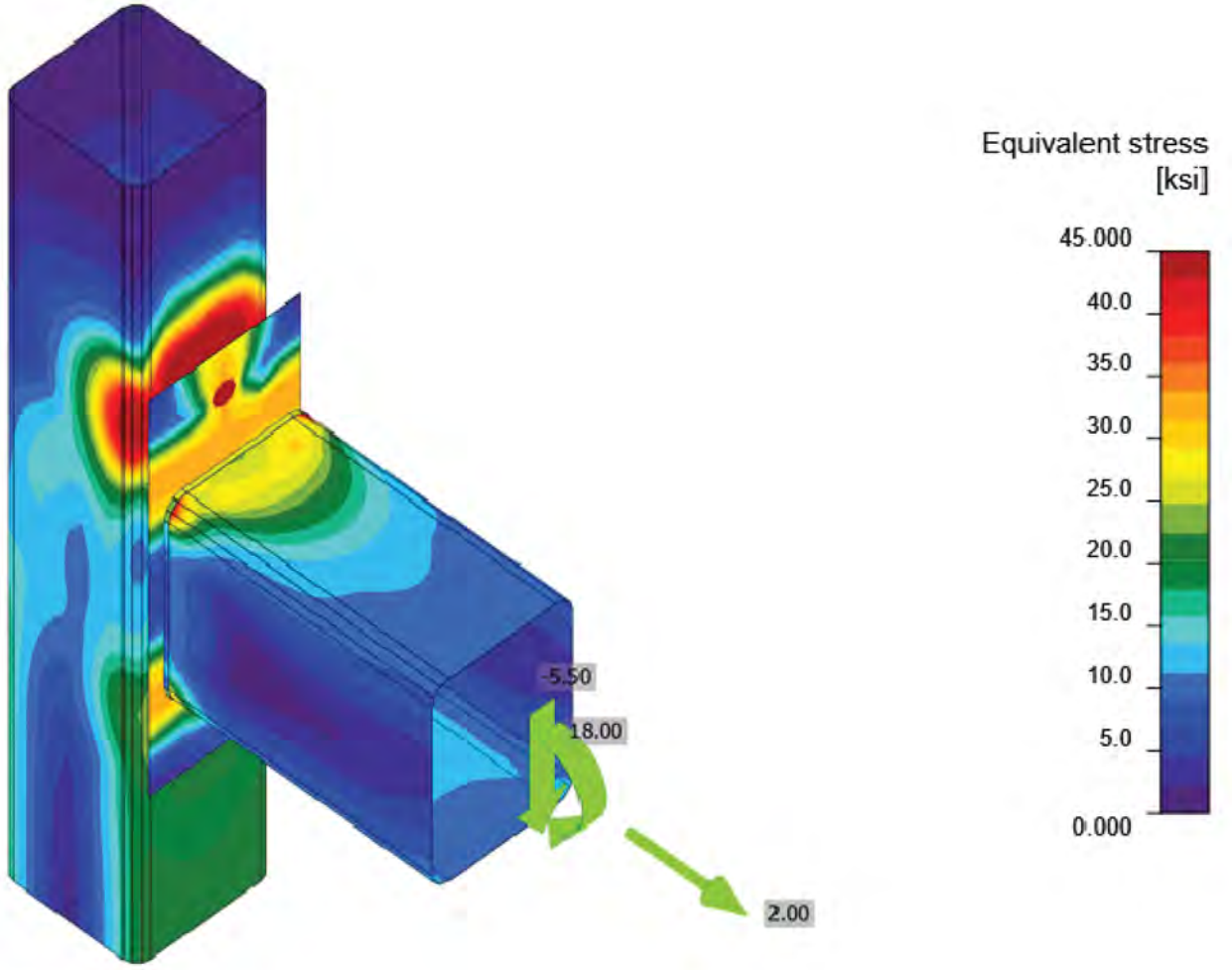


Overall check





Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B1	3/4 A325 - 1	LE1	25.00	2.62	41.85	83.91	14.68	0.00	OK	OK
	B2	3/4 A325 - 1	LE1	1.06	2.88	41.85	3.56	16.08	0.00	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
3/4 A325 - 1	29.79	17.88

Detailed result for B1

Tension resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = \quad 29.79 \quad \text{kip} \quad \geq \quad F_t = \quad 25.00 \quad \text{kip}$$

Where:

$F_{nt} = 89.92$ ksi – nominal tensile stress AISC 360-16 – Table J3.2

$A_b = 0.44$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nv} \cdot A_b = \quad 17.88 \quad \text{kip} \quad \geq \quad V = \quad 2.62 \quad \text{kip}$$

Where:

$F_{nv} = 53.95$ ksi – nominal shear stress AISC 360-16 – Table J3.2

$A_b = 0.44$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

Decisive plate: C-Web

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = \quad 41.85 \quad \text{kip} \quad \geq \quad V = \quad 2.62 \quad \text{kip}$$

Where:

$l_c = 9.19$ in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = 0.50$ in – thickness of the plate

$d = 0.75$ in – diameter of a bolt

$F_u = 62.00$ ksi – specified minimum tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B2

Tension resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = \quad 29.79 \quad \text{kip} \quad \geq \quad F_t = \quad 1.06 \quad \text{kip}$$

Where:

$F_{nt} = 89.92$ ksi – nominal tensile stress AISC 360-16 – Table J3.2

$A_b = 0.44$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 17.88 kip ≥ $V =$ 2.88 kip

Where:

$F_{nv} = 53.95$ ksi – nominal shear stress AISC 360-16 – Table J3.2

$A_b = 0.44$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

Decisive plate: C-Web

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 41.85 kip ≥ $V =$ 2.88 kip

Where:

$l_c = 7.34$ in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

$t = 0.50$ in – thickness of the plate

$d = 0.75$ in – diameter of a bolt

$F_u = 62.00$ ksi – specified minimum tensile strength of the connected material

$\phi = 0.75$ – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing	Status
EP1	B	E70xx	Ч 0.31	-	21.41	0.29	LE1	2.48	2.90	85.31	85.31	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.00

Detailed result for EP1 / B

Weld resistance check (AISC 360-16 – Table J2.5)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.90 kip ≥ $F_n =$ 2.48 kip

Where:

$F_{nw} =$ 42.00 ksi – nominal stress of weld material:

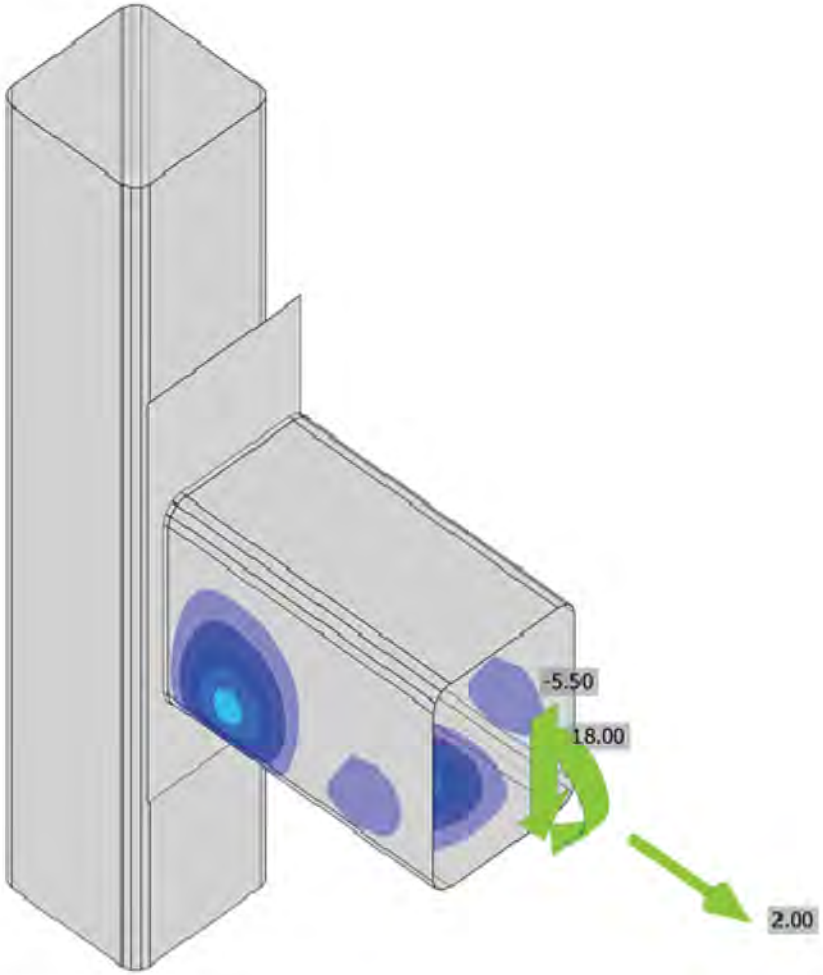
- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70.00 ksi – electrode classification number, i.e. minimum specified tensile strength

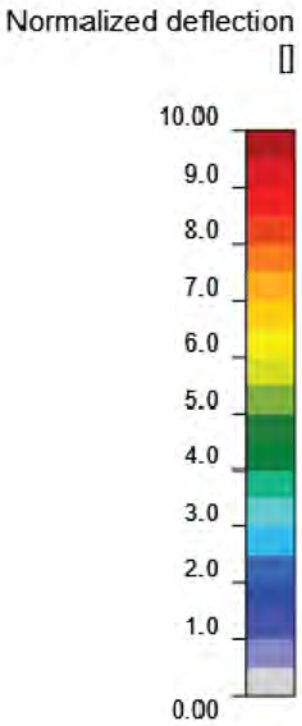
$A_{we} =$ 0.09 in^2 – effective area of weld critical element

$\phi =$ 0.75 – resistance factor for welded connections

Buckling

Loads	Shape	Factor [-]
LE1	1	68.96
	2	89.51
	3	93.11
	4	93.37
	5	108.31
	6	120.89





First buckling mode shape, LE1

Symbol explanation

Symbol	Explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation
δ/d_0	Relative deformation
F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6
t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
Ut_c	Weld capacity estimation
Ч	Partial joint penetration groove weld
F_{exx}	Ultimate strength as rated by electrode classification number

Code settings

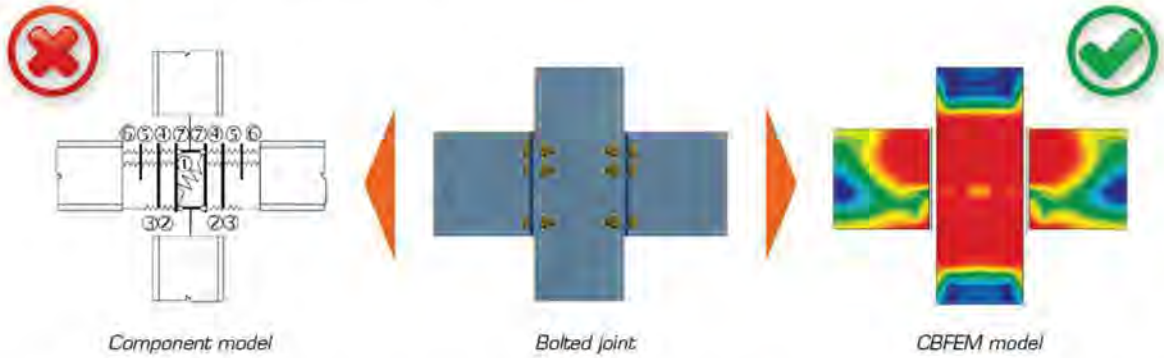
Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10

Item	Value	Unit	Reference
Cracked concrete	Yes		ACI 318-19 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 – 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Theoretical Background

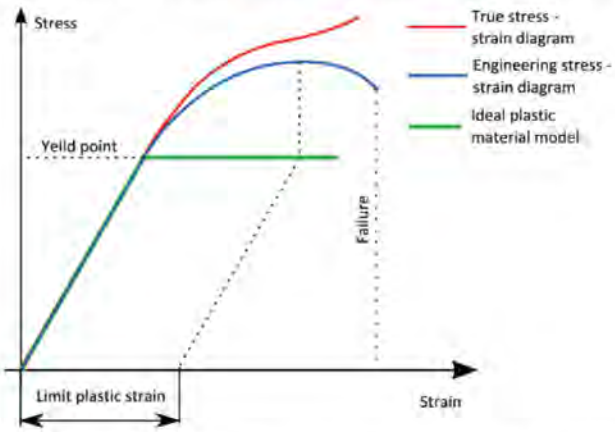
CBFEM versus AISC 360

The weak point of standard design method is in analyzing of internal forces and stress in a joint. CBFEM replaces specific analysis of internal forces in joint with general FEA.



Component model *Bolted joint* *CBFEM model*

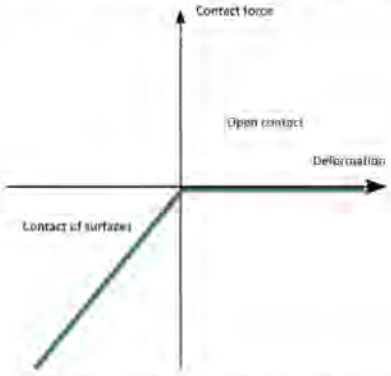
Check methods of specific components like bolts or welds are done according to standard AISC 360.
For the fasteners – bolts and welds – special FEM components had to be developed to model the welds and bolts behaviour in the connection. All parts of 1D members and all additional plates are modeled as plate/walls. These elements are made of steel (metal in general) and the behaviour of this material is significantly nonlinear.
The real stress-strain diagram of steel is replaced by the ideal plastic material for design purposes in building practice. The advantage of ideal plastic material is, that only yield strength and modulus of elasticity must be known to describe the material curve. The yield strength is multiplied by resistance factor (LRFD) or divided by safety factor (ASD) – AISC 360, Appendix 1. The granted ductility of construction steel is 15 %. The real usable value of limit plastic strain is 5% for ordinary design (EN 1993-1-5:2006 appendix C paragraph C.8 note 1).
The stress in steel cannot exceed the yield strength when using the ideal elastic-plastic stress-strain diagram.



Real tension curve and the ideal elastic-plastic diagram of material

CBFEM method aims to model the real state precisely. Meshes of plates / walls are not merged, no intersections are generated between them, unlike it is used to when modeling structures and buildings. Mesh of finite elements is generated on each individual

plate independently on mesh of other plates.
Between the meshes, special massless force interpolation constraints are added. They ensure the connection between the edge of one plate and the surface or edge of the other plate.
This unique calculation model provides very good results – both for the point of view of precision and of the analysis speed. The method is protected by patent.
The steel base plate is placed loosely on the concrete foundation. It is a contact element in the analysis model – the connection resists compression fully, but does not resist tension.



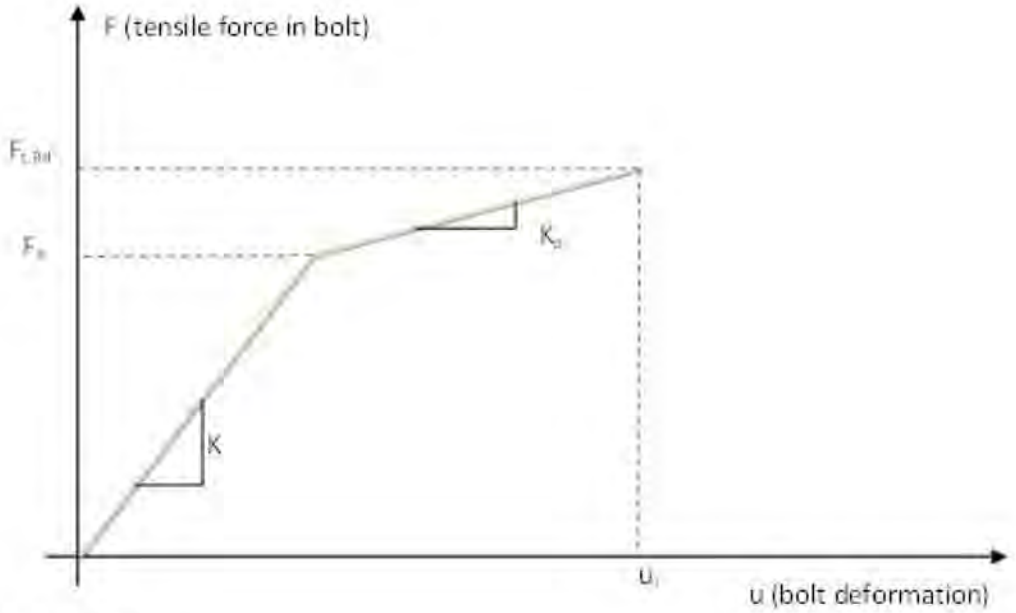
Stress-strain diagram of contact between the concrete block and the base plate

The concrete block in CBFEM is modeled using Winkler-Pasternak subsoil model. The stiffness of subsoil is determined using modulus of elasticity of concrete and effective height of subsoil. The concrete block is not designed by CBFEM method.

Welds are modeled using a special elastoplastic element, which is added to the interpolation links between the plates. The element respects the weld throat thickness, position and orientation. The plasticity state is controlled by stresses in the weld throat section. The plastic redistribution of stress in welds allows for stress peaks to be redistributed along the longer part of the weld.

Bolted connection consists of two or more clasped plates and one or more bolts. Plates are placed loosely on each other.
A contact element is inserted between plates in the analysis model, which acts only in compression. No forces are carried in tension.

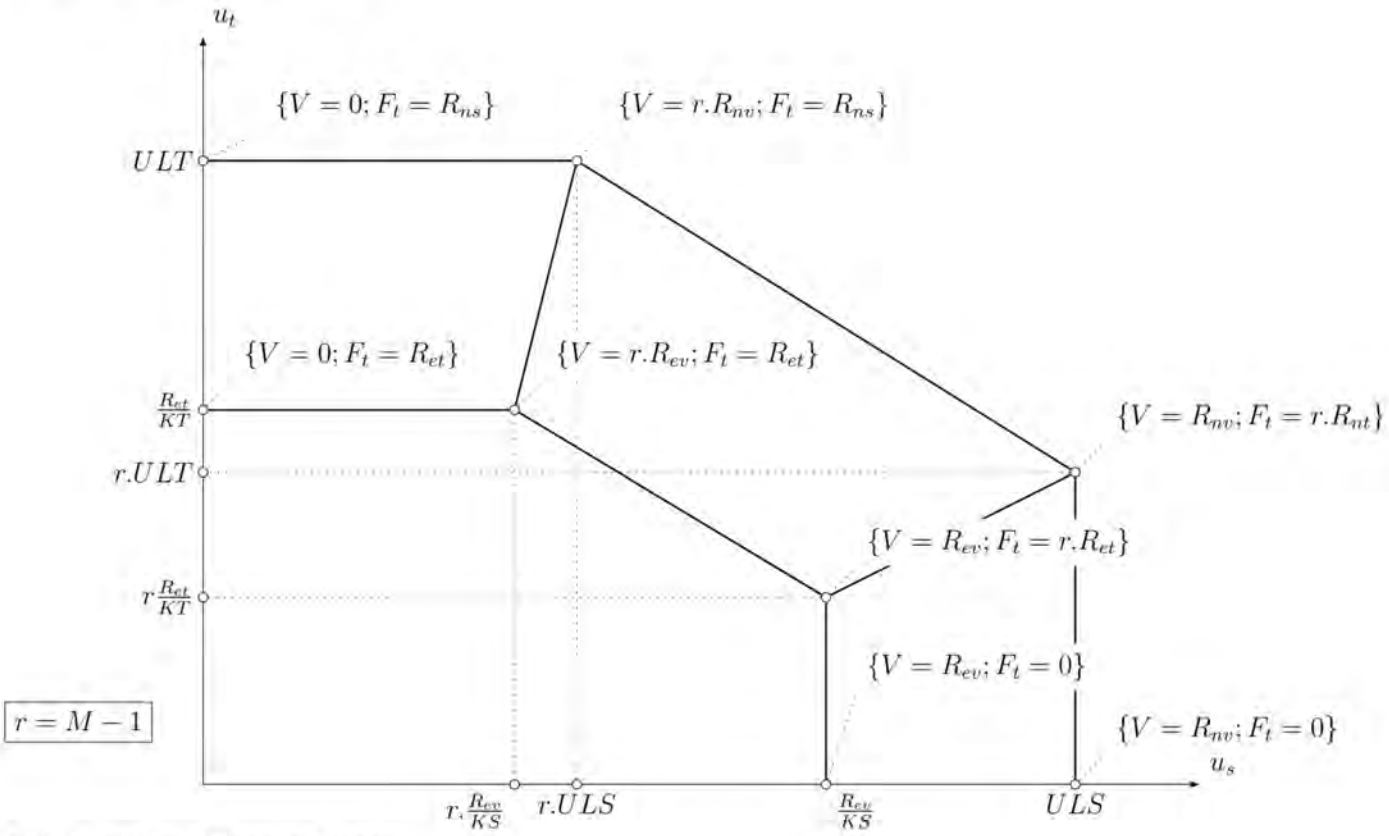
Shear force is taken by bearing. Special model for its transferring in the force direction only is implemented. IDEA StatiCa Connection can check bolts for interaction of shear and tension. The bolt behavior is implemented according to the following picture.



Bolt – tension

Symbols explanation:

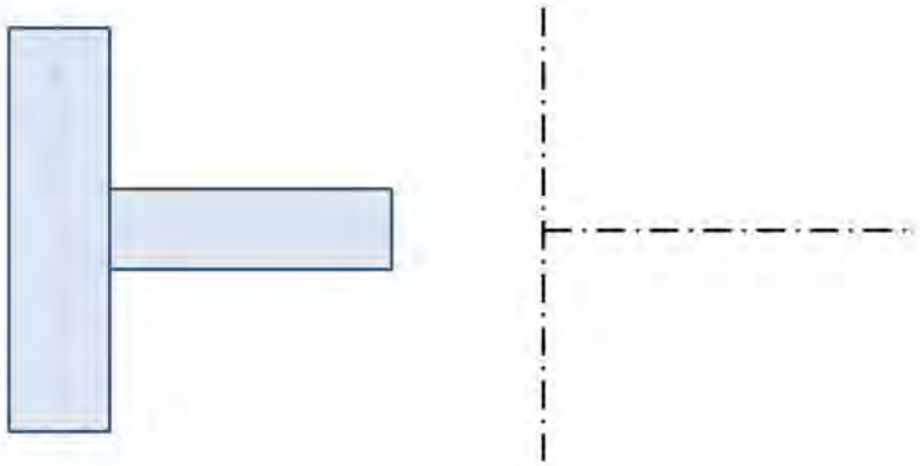
- K – linear stiffness of bolt,
- K_p – stiffness of bolt at plastic branch,
- F_{lt} – limit force for linear behaviour of bolt,
- $F_{t,Rd}$ – limit bolt resistance,
- u_l – limit deformation of bolt.



Bolt – interaction of shear and tension

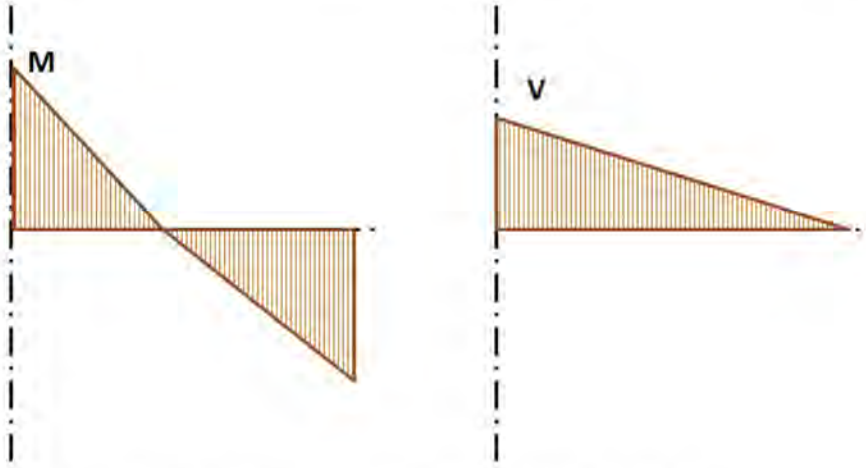
Loads

End forces of member of the frame analysis model are transferred to the ends of member segments. Eccentricities of members caused by the joint design are respected during load transfer.
The analysis model created by CBFEM method corresponds to the real joint very precisely, whereas the analysis of internal forces is performed on very idealised 3D FEM bar model, where individual beams are modeled using centrelines and the joints are modeled using immaterial nodes.



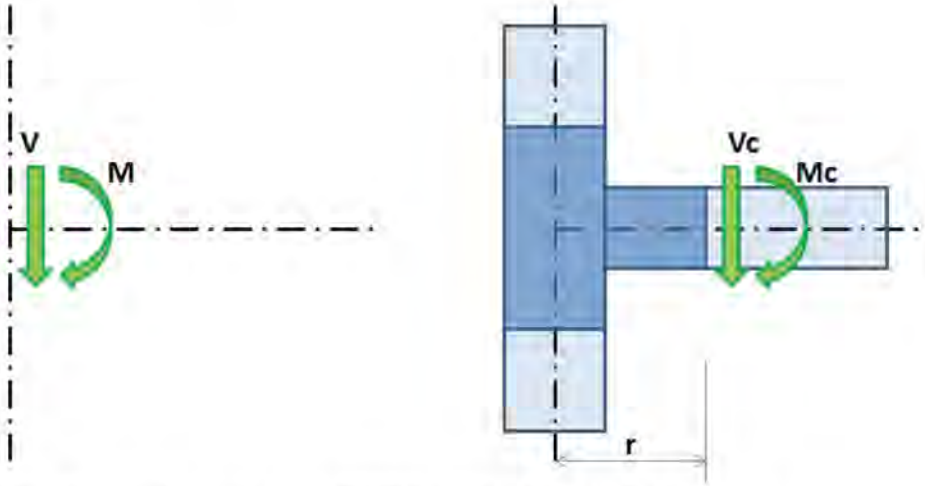
Joint of a vertical column and a horizontal beam

Internal forces are analysed using 1D members in 3D model. There is an example of courses of internal forces in the following picture.



Internal forces in horizontal beam. M and V are the end forces at joint.

The effects caused by member on the joint are important to design the connection. The effects are illustrated in the following picture.



Effects of the member on the joint. CBFEM model is drawn in dark blue color.

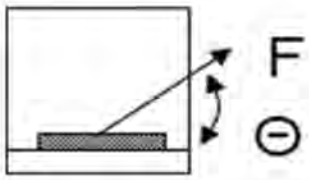
Moment M and shear force V act in a theoretical joint. The point of theoretical joint does not exist in CBFEM model, thus the load cannot be applied here. The model must be loaded by actions M and V, which have to be transferred to the end of segment in the distance r.
$M_c = M - V \cdot r$
$V_c = V$
In CBFEM model, the end section of segment is loaded by moment M_c and force V_c.

Welds

Fillet welds

The design strength, ϕR_n and the allowable strength, R_n/Ω of welded joints are evaluated in connection weld check.
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)
Available strength of welded joints is evaluated according to AISC 360 – J2.4:
$R_n = F_{nw} A_{we}$
$F_{nw} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \Theta)$
where

- F_{nw} – nominal stress of weld material,
- A_{we} – effective area of the weld,
- F_{EXX} – electrode classification number, i.e., minimum specified tensile strength,
- Θ – angle of loading measured from the weld longitudinal axis.



For long welds and welding to unstiffened flanges or webs of rectangular hollow sections, the weld material model is fine-tuned so that no reduction factor is necessary. The weld resistance is governed by most stressed weld element.

CJP groove welds

AISC Specification Table J2.5 identifies four loading conditions that might be associated with JP groove welds, and shows that the strength of the joint is either controlled by the base metal or that the loads need not be considered in the design of the welds connecting the parts. Accordingly, when CJP groove welds are made with matching-strength filler metal, the strength of a connection is governed or controlled by the base metal, and no checks on the weld strength are required.

Bolts

Tensile and shear strength of bolts

The design tensile or shear strength, ϕR_n, and the allowable tensile or shear strength, R_n / Ω of a snug-tightened bolt is determined according to the limit states of tension rupture and shear rupture as follows:
$R_n = F_n A_b$
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)
where

- A_b – nominal unthreaded body area of bolt or threaded part,
- F_n – nominal tensile stress, F_{nt}, or shear stress, F_{nv}, from Table J3.2.

The tensile force, against which the required tensile strength is checked, includes any tension resulting from prying action produced by deformation of the connected parts.

Combined Tension and shear in bearing type connection

The available tensile strength of a bolt subjected to combined tension and shear is determined according to the limit states of tension and shear rupture as follows:
$R_n = F'_{nt} A_b$ (AISC 360 J3-2)
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)
$F'_{nt} = 1.3 F_{nt} - f_{rv} F_{nt} / \phi F_{nv}$ (AISC 360 J3-3a LRFD)
$F'_{nt} = 1.3 F_{nt} - f_{rv} \Omega F_{nt} / F_{nv}$ (AISC 360 J3-3b ASD)
where

- F'_{nt} – nominal tensile stress modified to include the effects of shear stress,
- F_{nt} – nominal tensile stress from AISC 360 – Tab. J3.2,
- F_{nv} – nominal shear stress from AISC 360 – Tab. J3.2,
- f_{rv} – required shear stress using LRFD or ASD load combinations. The available shear stress of the fastener shall be equal or exceed the required shear stress, f_{rv}.

Bearing strength in bolt holes

The available bearing strength, ϕR_n and R_n/Ω at bolt holes is determined for the limit state of bearing as follows:
For a bolt in a connection with standard holes:
$R_n = 1.2 l_c t F_u \leq 2.4 d t F_u$ (AISC 360 J3-6a, c)
For a bolt in a connection with slotted holes:
$R_n = 1.0 l_c t F_u \leq 2.0 d t F_u$ (AISC 360 J3-6e, f)
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)

where

- F_u – specified minimum tensile strength of the connected material,
- d – nominal bolt diameter,
- l_c – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material,
- t – thickness of connected material.

Preloaded bolts

The design slip resistance of a preloaded class A325 or A490 bolt with of effect of tensile force, $F_{t,Ed}$ according to AISC 360 – J3.9.
Preloading force to be used AISC 360 – Tab. J3.1.
$T_b = 0.7 f_{ub} A_s$
Design slip resistance per bolt AISC 360 – J3.8
$R_n = 1.13 \mu T_b N_s$
Utilisation in shear [%]:
$U_{ts} = V / R_n$
where

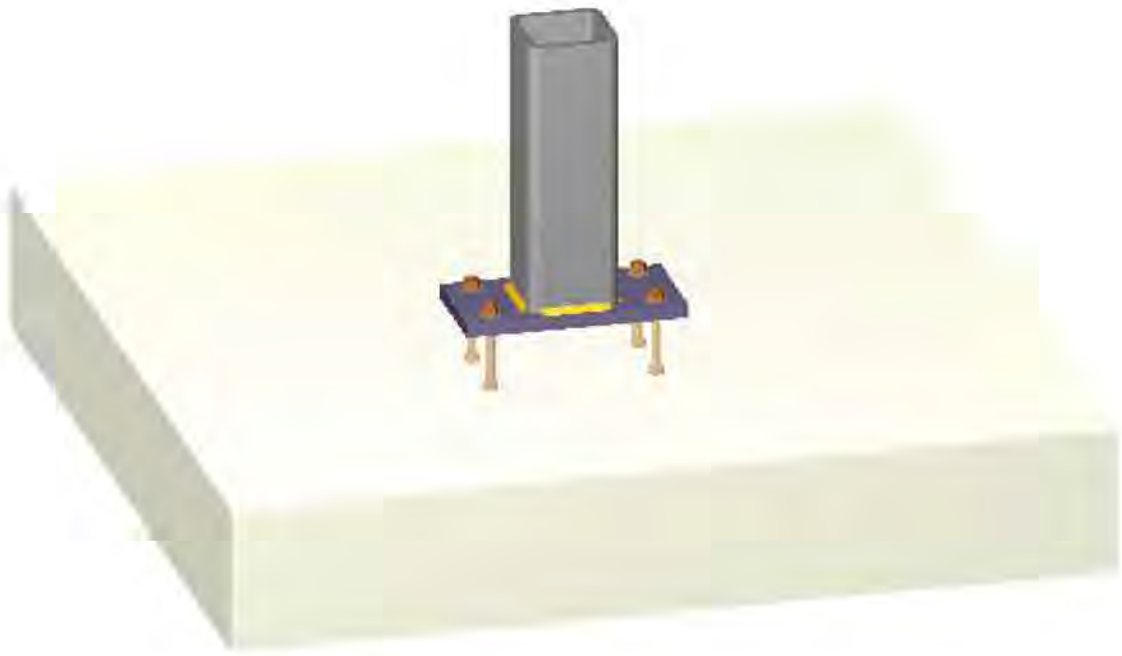
- A_s – tensile stress area of the bolt,

- f_{ub} – ultimate tensile strength,
- μ – mean slip factor coefficient,
- N_s – number of the friction surfaces. Check is calculated for each friction surface separately,
- V – shear force.

Anchors

The anchor bolt element is elastic-plastic with significant strain hardening. The maximum steel tensile resistance is expected at the strain which equals to 0.25 × guaranteed elongation. The failure mode due to concrete cracking may occur before the anchor steel tensile resistance is reached and is considered as a completely brittle failure.
Similarly, the steel components in shear (anchor bolt, base plate in bearing) are able to yield but failure modes connected with concrete cracking may occur suddenly as a brittle failure.
All standards use Concrete Capacity Design method developed by prof. R. Eligehausen at University of Stuttgart. The theory is based on vast experimental and numerical testing mostly on unreinforced concrete blocks and relatively short, often post-installed, anchors.
Anchorage is designed according to ACI 318 – Chapter 17. The design is available only for LRFD. Some failure modes (e.g. steel resistance) are evaluated for single anchors, others (e.g. concrete breakout) are checked for group of anchors.

MEP Equipment Support Frames between Grids B/F and 3/17
Base Plate



Steel Connections Results

Connection: 3 - Pinned BP

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 3

Design code: AISC 360-16 LRFD, ACI 318-19

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL1	-3.26	0.00	0.00	-0.01	0.00	Design
DL2	-19.11	0.00	0.00	-0.09	0.01	Design
E1	5.96	0.00	0.00	-1.34	0.00	Design
E2	-0.68	0.00	0.00	-0.01	-1.39	Design
D1	-31.33	0.00	0.00	-0.14	0.02	Design
D2	-20.50	0.00	0.00	-1.38	0.02	Design
D3	-27.61	0.00	0.00	-0.13	-1.28	Design
D4	-13.89	0.00	0.00	-1.37	0.01	Design
D5	-20.87	0.00	0.00	-0.10	-1.31	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]	0.75	0.25	--	✓	
Weld size	[1/16in]	5	3	--	✓	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	4.42	0.44	D1	0.10	DG1 3.1.1

Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	4.56	2.86	D1	0.63	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	4.56	1.05	E1	0.23	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	125.29	4.45	E1	0.04	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	83.53	5.43	E1	0.07	Sec. J2.4
Ratio	**0.63**					

Anchors

Geometric Considerations

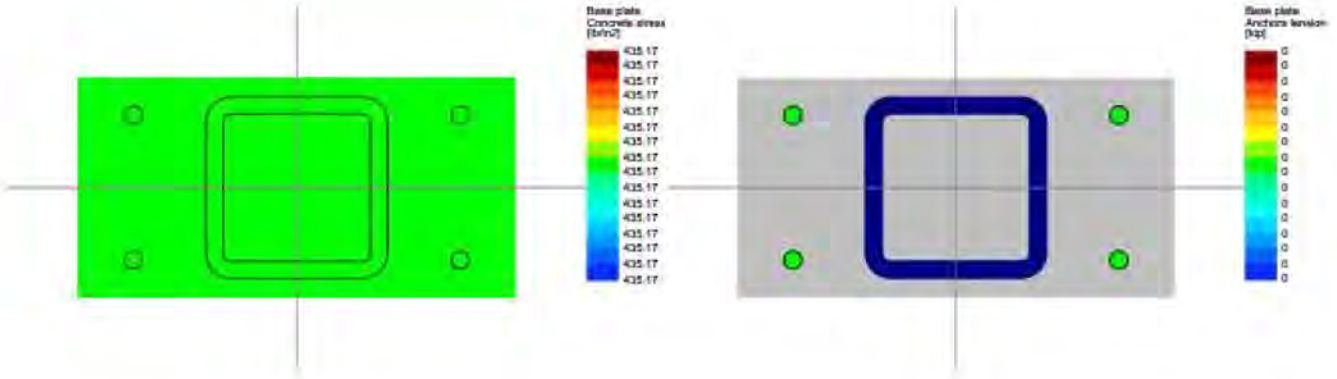
Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	4.00	2.00	--	✓	Sec. 17.9.2
Concrete cover	[in]	19.25	3.00	--	✓	Sec. 20.5.1.3.1
Effective length	[in]	3.33	--	7.67	✓	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	6.17	1.49	E1	0.24	Eq. 17.6.1.2
Breakout of anchor in tension	[Kip]	5.52	1.49	E1	0.27	Sec. 17.5.2
Breakout of group of anchors in tension	[Kip]	15.95	5.96	E1	0.37	Sec. 17.5.2
Pullout of anchor in tension	[Kip]	6.52	1.49	E1	0.23	Sec. 17.5.2
Anchor shear	[Kip]	3.21	0.34	D2	0.11	Eq. 17.7.1.2b
Breakout of anchor in shear	[Kip]	10.49	0.34	D2	0.03	Sec. 17.5.2
Breakout of anchor in shear parallel to edge	[Kip]	23.17	0.34	D2	0.01	Sec. 17.5.2
Breakout of group of anchors in shear	[Kip]	11.59	1.38	D2	0.12	Sec. 17.5.2
Breakout of group of anchors in shear parallel to edge	[Kip]	23.17	1.38	D2	0.06	Sec. 17.5.2
Pryout of anchor in shear	[Kip]	11.04	0.34	D2	0.03	Sec. 17.5.2
Pryout of group of anchors in shear	[Kip]	31.90	1.38	D2	0.04	Sec. 17.5.2
Interaction of tensile and shear forces	[Kip]	1.20	0.00	E1	0.00	Sec. 17.8.1
Ratio	**0.37**					
Global critical strength ratio	**0.63**					

Major axis

Maximum compression (D1)

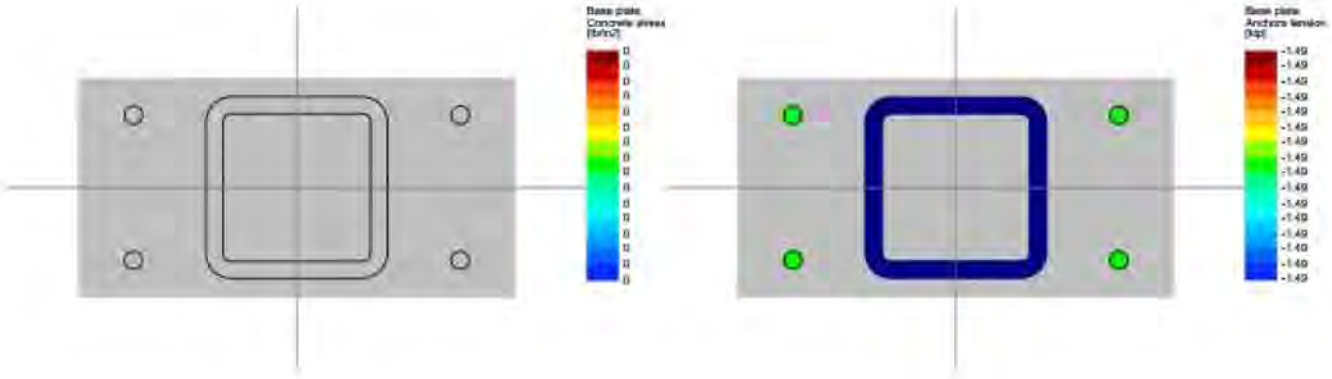


Maximum bearing pressure	435.17	[lb/in2]
Minimum bearing pressure	435.17	[lb/in2]
Maximum anchor tension	0.00	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	1.2E31	[in]
Bearing length	6.00	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.50	-2.00	-0.03	0.00
2	-4.50	2.00	-0.03	0.00
3	4.50	2.00	-0.03	0.00
4	4.50	-2.00	-0.03	0.00

Maximum tension (E1)



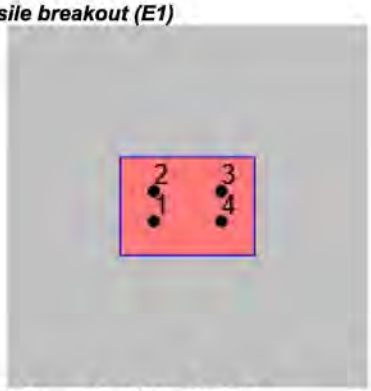
Maximum bearing pressure	0.00	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	1.49	[kip]
Minimum anchor tension	1.49	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	-1.2E31	[in]
Bearing length	0.00	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.50	-2.00	-0.33	1.49
2	-4.50	2.00	-0.33	1.49
3	4.50	2.00	-0.33	1.49
4	4.50	-2.00	-0.33	1.49

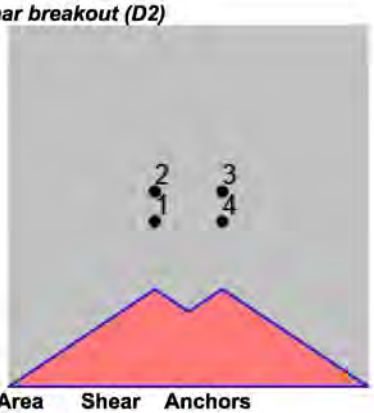
Major axis

Results for tensile breakout (E1)



Group	Area [in2]	Tension [kip]	Anchors
1	234.00	5.96	1, 2, 3, 4

Results for shear breakout (D2)



Group	Area [in2]	Shear [kip]	Anchors
1	384.00	0.69	1, 4
2	384.00	1.38	1, 2, 3, 4

MEP Equipment Support Frames between Grids B/F and 3/17
Thickened Slab on Grade

Combined Footing

Project File: SOG Under HAC Columns.ec6

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Code References

Governing Code : IBC 2021
Referenced Design Standard(s) : ACI 318-19
Load Combinations Used : ASCE 7-22 / IBC 2024 Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength		3.0 ksi
fy : Rebar Yield		60.0 ksi
Ec : Concrete Elastic Modulus		3,122.0 ksi
Concrete Density		145.0 pcf
ϕ : Phi Values	Flexure :	0.90
	Shear :	0.750

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel Ratio per ACI 7.6.1.1 (ratio=Steel Area/Concrete Area)	0.0018
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	2.50 ksf
Increase Bearing By Footing Weight	No
Soil Passive Sliding Resistance	250.0 pcf
(Uses entry for "Footing base depth below soil surface" for force)	
Coefficient of Soil/Concrete Friction	0.30

Soil Bearing Increase

Footing base depth below soil surface	ft
Increases based on footing Depth	
Allowable pressure increase per foot	ksf
when base of footing is below	ft
Increases based on footing Width . . .	
Allowable pressure increase per foot	ksf
when maximum length or width is greater tha	ft
Maximum Allowed Bearing Pressure	10.0 ksf
(A value of zero implies no limit)	
Adjusted Allowable Soil Bearing	2.50 ksf
(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)	

Dimensions & Reinforcing

Distance Left of Column #1	=	2.0 ft
Between Columns	=	6.333 ft
Distance Right of Column #2	=	2.0 ft
Total Footing Length	=	10.333 ft
Footing Width	=	6.0 ft
Footing Thickness	=	8.0 in
Rebar Center to Concrete Edge @ Top	=	1.50 in
Rebar Center to Concrete Edge @ Bottom	=	3.0 in

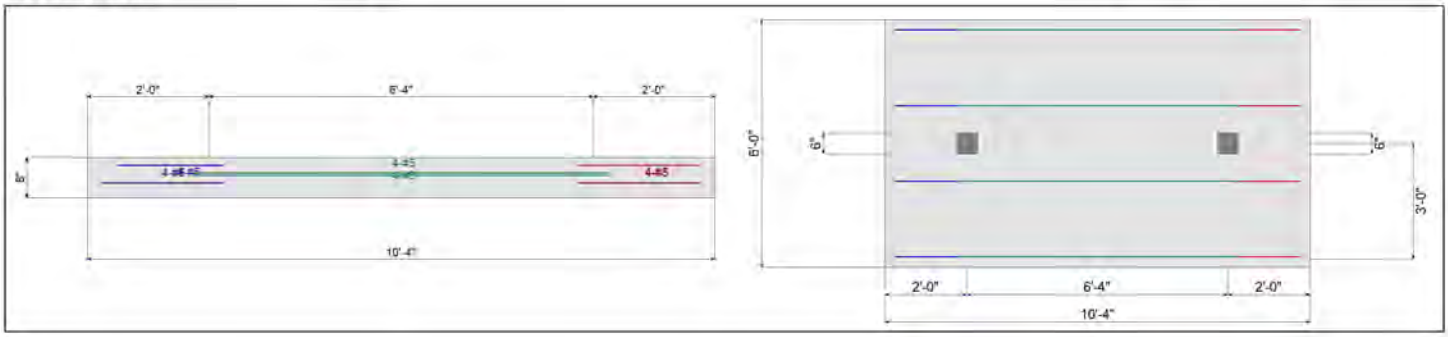
Pedestal dimensions...

		Col #1	Col #2	
Sq. Dim.	=	6.0	6.0	in
Height	=			in

	Count	Size #	As Provided
Bars left of Col #1			
Bottom Bars	4.0	5	1.240
Top Bars	4.0	5	1.240
Bars Btwn Cols			
Bottom Bars	4.0	5	1.240
Top Bars	4.0	5	1.240
Bars Right of Col #2			
Bottom Bars	4.0	5	1.240
Top Bars	4.0	5	1.240

Applied Loads

Applied @ Left Column		D	Lr	L	S	W	E	H	
Axial Load Downward	=	11.670					6.080		k
Moment (+CW)	=								k-ft
Shear (+X)	=						1.240		k
Applied @ Right Column									
Axial Load Downward	=	22.370					-5.960		k
Moment (+CW)	=								k-ft
Shear (+X)	=						1.340		k
Overburden	=								



Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: SOG Under HAC Columns.ec6

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

DESIGN SUMMARY

Design OK

Factor of Safety		Item	Applied	Capacity	Governing Load Combination
PASS	4.434	Overturning	34.765 k-ft	154.144 k-ft	+0.60D+0.70E
PASS	4.004	Sliding	1.806 k	7.231 k	+0.60D+0.70E
PASS	6.778	Uplift	4.172 k	28.276 k	+0.60D+0.70E

Utilization Ratio		Item	Applied	Capacity	Governing Load Combination
PASS	0.3849	Soil Bearing	0.9623 ksf	2.50 ksf	D Only
PASS	0.5704	1-way Shear - Col #1	25.937 psi	45.475 psi	+1.40D
PASS	0.7715	1-way Shear - Col #2	38.292 psi	49.631 psi	+1.40D
PASS	0.5413	2-way Punching - Col #1	88.951 psi	164.317 psi	+1.20D+E
PASS	0.5402	2-way Punching - Col #2	88.771 psi	164.317 psi	+1.20D+E
PASS	No Tension	Flexure - Left of Col #1 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.210	Flexure - Left of Col #1 - Bottom	5.620 k-ft	26.769 k-ft	+1.20D+E
PASS	0.3966	Flexure - Between Cols - Top	-13.937 k-ft	35.139 k-ft	+1.40D
PASS	0.3450	Flexure - Between Cols - Bottom	9.236 k-ft	26.769 k-ft	+1.40D
PASS	No Tension	Flexure - Right of Col #2 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.3905	Flexure - Right of Col #2 - Bottom	10.455 k-ft	26.769 k-ft	+1.40D

Soil Bearing

Load Combination...	Total Bearing	Eccentricity from Ftg CL	Actual Soil Bearing Stress @ Left Edge	@ Right Edge	Allowable	Actual / Allow Ratio
D Only	40.03 k	0.846 ft	0.33 ksf	0.96 ksf	2.50 ksf	0.385
+0.60D	24.02 k	0.846 ft	0.20 ksf	0.58 ksf	2.50 ksf	0.231
+D+0.70E	40.12 k	0.209 ft	0.57 ksf	0.73 ksf	2.50 ksf	0.290
+D+0.5250E	40.10 k	0.368 ft	0.51 ksf	0.78 ksf	2.50 ksf	0.314
+0.60D+0.70E	24.10 k	-0.214 ft	0.44 ksf	0.34 ksf	2.50 ksf	0.175

Overturning Stability

Load Combination...	Moments about Left Edge k-ft Overturning	Resisting	Ratio	Moments about Right Edg k-ft Overturning	Resisting	Ratio
D Only	0.00	0.00	999.000	0.00	0.00	999.000
+0.60D	0.00	0.00	999.000	0.00	0.00	999.000
+D+0.70E	34.77	250.43	7.203	9.55	208.41	21.828
+D+0.5250E	26.07	248.00	9.511	7.16	199.55	27.866
+0.60D+0.70E	34.77	154.14	4.434	9.55	139.24	14.583

Sliding Stability

Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio
D Only	0.00 k	12.01 k	999
+0.60D	0.00 k	7.21 k	999
+D+0.70E	1.81 k	12.04 k	6.664
+D+0.5250E	1.35 k	12.03 k	8.881
+0.60D+0.70E	1.81 k	7.23 k	4.004

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.60D+0.70E	0.000	0.000	0	0.000	0	0.000	0.000	0.000
+0.60D+0.70E	0.000	0.026	0	0.000	0	0.000	0.000	0.000
+0.60D+0.70E	0.000	0.052	0	0.000	0	0.000	0.000	0.000
+1.20D+E	0.011	0.077	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.000
+1.20D+E	0.020	0.103	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.001
+1.20D+E	0.031	0.129	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.001
+1.20D+E	0.045	0.155	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.002
+1.20D+E	0.061	0.181	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.002
+1.20D+E	0.080	0.207	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.003
+1.20D+E	0.101	0.232	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.004
+1.20D+E	0.124	0.258	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.005
+1.20D+E	0.151	0.284	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.006

Combined Footing

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+E	0.179	0.310	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.007
+1.20D+E	0.210	0.336	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.008
+1.20D+E	0.244	0.362	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.009
+1.20D+E	0.280	0.387	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.010
+1.20D+E	0.319	0.413	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.012
+1.20D+E	0.360	0.439	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.013
+1.20D+E	0.404	0.465	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.015
+1.20D+E	0.450	0.491	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.017
+1.20D+E	0.498	0.517	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.019
+1.20D+E	0.549	0.542	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.021
+1.20D+E	0.603	0.568	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.023
+1.20D+E	0.659	0.594	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.025
+1.20D+E	0.718	0.620	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.027
+1.20D+E	0.779	0.646	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.029
+1.20D+E	0.843	0.672	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.031
+1.20D+E	0.909	0.697	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.034
+1.20D+E	0.978	0.723	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.037
+1.20D+E	1.049	0.749	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.039
+1.20D+E	1.122	0.775	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.042
+1.20D+E	1.199	0.801	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.045
+1.20D+E	1.277	0.827	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.048
+1.20D+E	1.359	0.852	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.051
+1.20D+E	1.442	0.878	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.054
+1.20D+E	1.529	0.904	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.057
+1.20D+E	1.617	0.930	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.060
+1.20D+E	1.709	0.956	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.064
+1.20D+E	1.802	0.982	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.067
+1.20D+E	1.899	1.007	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.071
+1.20D+E	1.998	1.033	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.075
+1.20D+E	2.099	1.059	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.078
+1.20D+E	2.203	1.085	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.082
+1.20D+E	2.309	1.111	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.086
+1.20D+E	2.418	1.137	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.090
+1.20D+E	2.529	1.162	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.094
+1.20D+E	2.643	1.188	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.099
+1.20D+E	2.760	1.214	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.103
+1.20D+E	2.879	1.240	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.108
+1.20D+E	3.000	1.266	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.112
+1.20D+E	3.124	1.292	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.117
+1.20D+E	3.251	1.317	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.121
+1.20D+E	3.380	1.343	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.126
+1.20D+E	3.512	1.369	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.131
+1.20D+E	3.646	1.395	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.136
+1.20D+E	3.783	1.421	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.141
+1.20D+E	3.922	1.447	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.147
+1.20D+E	4.064	1.472	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.152
+1.20D+E	4.208	1.498	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.157
+1.20D+E	4.355	1.524	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.163
+1.20D+E	4.504	1.550	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.168
+1.20D+E	4.656	1.576	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.174
+1.20D+E	4.810	1.602	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.180
+1.20D+E	4.967	1.627	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.186
+1.20D+E	5.127	1.653	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.192
+1.20D+E	5.289	1.679	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.198
+1.20D+E	5.453	1.705	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.204
+1.20D+E	5.620	1.731	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.210
+1.20D+E	5.789	1.757	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.216
+1.20D+E	5.941	1.782	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.222
+1.20D+E	6.069	1.808	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.227
+1.20D+E	6.172	1.834	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.231
+1.20D+E	6.251	1.860	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.234
+1.20D+E	6.306	1.886	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.236
+1.20D+E	6.337	1.912	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.237

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: SOG Under HAC Columns.ec6

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+E	6.343	1.937	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.237
+1.20D+E	6.325	1.963	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.236
+1.20D+E	6.283	1.989	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.235
+1.20D+E	7.043	2.015	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.263
+1.20D+E	6.953	2.041	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.260
+1.20D+E	6.838	2.067	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.255
+1.20D+E	6.698	2.092	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.250
+1.20D+E	6.535	2.118	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.244
+1.20D+E	6.347	2.144	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.237
+1.20D+E	6.135	2.170	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.229
+1.20D+E	5.899	2.196	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.220
+1.20D+E	5.639	2.222	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.211
+1.20D+E	5.354	2.247	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.200
+1.20D+E	5.056	2.273	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.189
+1.20D+E	4.760	2.299	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.178
+1.20D+E	4.467	2.325	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.167
+1.20D+E	4.177	2.351	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.156
+1.20D+E	3.889	2.377	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.145
+1.20D+E	3.603	2.402	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.135
+0.90D+E	3.326	2.428	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.124
+0.90D+E	3.101	2.454	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.116
+0.90D+E	2.879	2.480	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.108
+0.90D+E	2.658	2.506	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.099
+0.90D+E	2.440	2.532	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.091
+0.90D+E	2.224	2.557	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.083
+0.90D+E	2.010	2.583	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.075
+0.90D+E	1.797	2.609	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.067
+0.90D+E	1.588	2.635	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.059
+1.40D	-2.285	2.661	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.065
+1.40D	-2.525	2.687	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.072
+1.40D	-2.763	2.712	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.079
+1.40D	-2.999	2.738	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.085
+1.40D	-3.232	2.764	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.092
+1.40D	-3.463	2.790	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.099
+1.40D	-3.692	2.816	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.105
+1.40D	-3.918	2.842	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.112
+1.40D	-4.143	2.867	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.118
+1.40D	-4.364	2.893	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.124
+1.40D	-4.584	2.919	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.130
+1.40D	-4.801	2.945	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.137
+1.40D	-5.016	2.971	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.143
+1.40D	-5.229	2.997	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.149
+1.40D	-5.440	3.022	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.155
+1.40D	-5.648	3.048	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.161
+1.40D	-5.853	3.074	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.167
+1.40D	-6.057	3.100	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.172
+1.40D	-6.258	3.126	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.178
+1.40D	-6.456	3.152	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.184
+1.40D	-6.652	3.177	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.189
+1.40D	-6.846	3.203	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.195
+1.40D	-7.038	3.229	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.200
+1.40D	-7.227	3.255	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.206
+1.40D	-7.413	3.281	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.211
+1.40D	-7.597	3.307	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.216
+1.40D	-7.779	3.332	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.221
+1.40D	-7.958	3.358	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.226
+1.40D	-8.135	3.384	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.232
+1.40D	-8.309	3.410	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.236
+1.40D	-8.481	3.436	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.241
+1.40D	-8.651	3.462	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.246
+1.40D	-8.818	3.487	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.251
+1.40D	-8.982	3.513	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.256
+1.40D	-9.144	3.539	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.260

Combined Footing

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DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.40D	-9.303	3.565	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.265
+1.40D	-9.460	3.591	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.269
+1.40D	-9.614	3.617	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.274
+1.40D	-9.766	3.642	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.278
+1.40D	-9.915	3.668	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.282
+1.40D	-10.062	3.694	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.286
+1.40D	-10.206	3.720	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.290
+1.40D	-10.348	3.746	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.294
+1.40D	-10.487	3.772	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.298
+1.40D	-10.623	3.797	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.302
+1.40D	-10.757	3.823	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.306
+1.40D	-10.888	3.849	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.310
+1.40D	-11.016	3.875	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.314
+1.40D	-11.142	3.901	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.317
+1.40D	-11.265	3.927	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.321
+1.40D	-11.386	3.952	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.324
+1.40D	-11.504	3.978	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.327
+1.40D	-11.619	4.004	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.331
+1.40D	-11.731	4.030	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.334
+1.40D	-11.841	4.056	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.337
+1.40D	-11.948	4.082	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.340
+1.40D	-12.053	4.107	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.343
+1.40D	-12.155	4.133	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.346
+1.40D	-12.254	4.159	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.349
+1.40D	-12.350	4.185	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.351
+1.40D	-12.444	4.211	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.354
+1.40D	-12.534	4.237	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.357
+1.40D	-12.622	4.262	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.359
+1.40D	-12.708	4.288	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.362
+1.40D	-12.790	4.314	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.364
+1.40D	-12.870	4.340	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.366
+1.40D	-12.947	4.366	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.368
+1.40D	-13.021	4.392	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.371
+1.40D	-13.092	4.417	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.373
+1.40D	-13.161	4.443	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.375
+1.40D	-13.226	4.469	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.376
+1.40D	-13.289	4.495	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.378
+1.40D	-13.349	4.521	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.380
+1.40D	-13.406	4.547	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.382
+1.40D	-13.461	4.572	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.383
+1.40D	-13.512	4.598	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.385
+1.40D	-13.561	4.624	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.386
+1.40D	-13.606	4.650	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.387
+1.40D	-13.649	4.676	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.388
+1.40D	-13.689	4.702	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.390
+1.40D	-13.726	4.727	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.391
+1.40D	-13.760	4.753	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.392
+1.40D	-13.791	4.779	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.392
+1.40D	-13.819	4.805	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.393
+1.40D	-13.844	4.831	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.394
+1.40D	-13.866	4.857	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.395
+1.40D	-13.885	4.882	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.395
+1.40D	-13.901	4.908	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.915	4.934	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.925	4.960	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.932	4.986	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.936	5.012	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.397
+1.40D	-13.937	5.037	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.397
+1.40D	-13.936	5.063	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.397
+1.40D	-13.931	5.089	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.923	5.115	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.912	5.141	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396
+1.40D	-13.898	5.166	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.396

Combined Footing

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.40D	-13.881	5.192	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.395
+1.40D	-13.860	5.218	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.394
+1.40D	-13.837	5.244	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.394
+1.40D	-13.811	5.270	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.393
+1.40D	-13.781	5.296	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.392
+1.40D	-13.748	5.321	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.391
+1.40D	-13.713	5.347	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.390
+1.40D	-13.674	5.373	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.389
+1.40D	-13.631	5.399	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.388
+1.40D	-13.586	5.425	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.387
+1.40D	-13.538	5.451	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.385
+1.40D	-13.486	5.476	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.384
+1.40D	-13.431	5.502	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.382
+1.40D	-13.373	5.528	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.381
+1.40D	-13.312	5.554	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.379
+1.40D	-13.248	5.580	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.377
+1.40D	-13.180	5.606	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.375
+1.40D	-13.110	5.631	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.373
+1.40D	-13.035	5.657	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.371
+1.40D	-12.958	5.683	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.369
+1.40D	-12.878	5.709	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.366
+1.40D	-12.794	5.735	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.364
+1.40D	-12.707	5.761	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.362
+1.40D	-12.616	5.786	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.359
+1.40D	-12.523	5.812	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.356
+1.40D	-12.426	5.838	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.354
+1.40D	-12.325	5.864	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.351
+1.40D	-12.222	5.890	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.348
+1.40D	-12.115	5.916	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.345
+1.40D	-12.004	5.941	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.342
+1.40D	-11.891	5.967	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.338
+1.40D	-11.774	5.993	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.335
+1.40D	-11.654	6.019	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.332
+1.40D	-11.530	6.045	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.328
+1.40D	-11.403	6.071	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.325
+1.40D	-11.272	6.096	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.321
+1.40D	-11.138	6.122	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.317
+1.40D	-11.001	6.148	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.313
+1.40D	-10.860	6.174	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.309
+1.40D	-10.716	6.200	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.305
+1.40D	-10.569	6.226	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.301
+1.40D	-10.418	6.251	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.296
+1.40D	-10.263	6.277	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.292
+1.40D	-10.105	6.303	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.288
+1.40D	-9.944	6.329	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.283
+1.40D	-9.779	6.355	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.278
+1.40D	-9.611	6.381	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.274
+1.40D	-9.439	6.406	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.269
+1.40D	-9.264	6.432	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.264
+1.40D	-9.085	6.458	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.259
+1.40D	-8.903	6.484	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.253
+1.40D	-8.717	6.510	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.248
+1.20D+E	-8.543	6.536	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.243
+1.20D+E	-8.405	6.561	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.239
+1.20D+E	-8.266	6.587	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.235
+1.20D+E	-8.123	6.613	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.231
+1.20D+E	-7.978	6.639	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.227
+1.20D+E	-7.830	6.665	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.223
+1.20D+E	-7.680	6.691	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.219
+1.20D+E	-7.527	6.716	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.214
+1.20D+E	-7.371	6.742	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.210
+1.20D+E	-7.212	6.768	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.205
+1.20D+E	-7.051	6.794	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.201

Combined Footing

LIC# : KW-06018138, Build:20.26.07.20 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+E	-6.887	6.820	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.196
+1.20D+E	-6.720	6.846	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.191
+1.20D+E	-6.551	6.871	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.186
+1.20D+E	-6.379	6.897	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.182
+1.20D+E	-6.204	6.923	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.177
+1.20D+E	-6.026	6.949	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.171
+1.20D+E	-5.846	6.975	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.166
+1.20D+E	-5.663	7.001	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.161
+1.20D+E	-5.478	7.026	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.156
+1.20D+E	-5.289	7.052	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.151
+1.20D+E	-5.098	7.078	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.145
+1.20D+E	-4.905	7.104	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.140
+1.20D+E	-4.708	7.130	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.134
+1.20D+E	-4.509	7.156	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.128
+1.20D+E	-4.307	7.181	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.123
+1.20D+E	-4.103	7.207	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.117
+1.20D+E	-3.895	7.233	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.111
+1.20D+E	-3.685	7.259	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.105
+1.20D+E	-3.473	7.285	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.099
+1.20D+E	-3.257	7.311	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.093
+1.20D+E	-3.039	7.336	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.086
+1.20D+E	-2.819	7.362	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.080
+1.20D+E	-2.595	7.388	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.074
+0.90D+E	-2.387	7.414	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.068
+0.90D+E	-2.226	7.440	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.063
+0.90D+E	-2.064	7.466	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.059
+0.90D+E	-1.899	7.491	Top	1.037	ACI 7.6.1.1	1.240	35.139	0.054
+1.40D	1.383	7.517	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.052
+1.40D	1.717	7.543	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.064
+1.40D	2.056	7.569	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.077
+1.40D	2.398	7.595	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.090
+1.40D	2.744	7.621	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.103
+1.40D	3.094	7.646	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.116
+1.40D	3.448	7.672	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.129
+1.40D	3.806	7.698	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.142
+1.40D	4.168	7.724	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.156
+1.40D	4.534	7.750	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.169
+1.40D	4.904	7.776	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.183
+1.40D	5.277	7.801	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.197
+1.40D	5.655	7.827	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.211
+1.40D	6.037	7.853	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.226
+1.40D	6.423	7.879	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.240
+1.40D	6.813	7.905	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.254
+1.40D	7.206	7.931	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.269
+1.40D	7.604	7.956	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.284
+1.40D	8.006	7.982	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.299
+1.40D	8.412	8.008	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.314
+1.40D	8.822	8.034	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.330
+1.40D	9.236	8.060	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.345
+1.40D	9.654	8.086	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.361
+1.40D	10.051	8.111	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.375
+1.40D	10.411	8.137	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.389
+1.40D	10.732	8.163	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.401
+1.40D	11.016	8.189	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.412
+1.40D	11.263	8.215	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.421
+1.40D	11.472	8.241	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.429
+1.40D	11.643	8.266	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.435
+1.40D	11.776	8.292	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.440
+1.40D	11.872	8.318	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.443
+1.40D	11.930	8.344	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.446
+1.40D	11.950	8.370	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.446
+1.40D	11.933	8.396	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.446
+1.40D	11.878	8.421	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.444

Combined Footing

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DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.40D	11.786	8.447	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.440
+1.40D	11.656	8.473	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.435
+1.40D	11.489	8.499	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.429
+1.40D	11.284	8.525	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.422
+1.40D	11.041	8.551	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.412
+1.40D	10.761	8.576	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.402
+1.40D	10.455	8.602	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.391
+1.40D	10.151	8.628	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.379
+1.40D	9.852	8.654	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.368
+1.40D	9.557	8.680	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.357
+1.40D	9.267	8.706	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.346
+1.40D	8.981	8.731	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.335
+1.40D	8.699	8.757	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.325
+1.40D	8.421	8.783	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.315
+1.40D	8.148	8.809	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.304
+1.40D	7.879	8.835	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.294
+1.40D	7.615	8.861	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.284
+1.40D	7.355	8.886	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.275
+1.40D	7.099	8.912	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.265
+1.40D	6.847	8.938	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.256
+1.40D	6.600	8.964	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.247
+1.40D	6.358	8.990	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.237
+1.40D	6.119	9.016	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.229
+1.40D	5.885	9.041	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.220
+1.40D	5.656	9.067	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.211
+1.40D	5.431	9.093	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.203
+1.40D	5.210	9.119	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.195
+1.40D	4.994	9.145	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.187
+1.40D	4.782	9.171	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.179
+1.40D	4.575	9.196	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.171
+1.40D	4.372	9.222	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.163
+1.40D	4.174	9.248	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.156
+1.40D	3.980	9.274	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.149
+1.40D	3.790	9.300	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.142
+1.40D	3.605	9.326	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.135
+1.40D	3.425	9.351	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.128
+1.40D	3.249	9.377	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.121
+1.40D	3.078	9.403	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.115
+1.40D	2.911	9.429	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.109
+1.40D	2.749	9.455	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.103
+1.40D	2.591	9.481	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.097
+1.40D	2.438	9.506	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.091
+1.40D	2.289	9.532	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.086
+1.40D	2.145	9.558	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.080
+1.40D	2.006	9.584	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.075
+1.40D	1.871	9.610	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.070
+1.40D	1.741	9.636	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.065
+1.40D	1.615	9.661	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.060
+1.40D	1.494	9.687	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.056
+1.40D	1.378	9.713	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.051
+1.40D	1.266	9.739	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.047
+1.40D	1.159	9.765	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.043
+1.40D	1.057	9.791	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.039
+1.40D	0.959	9.816	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.036
+1.40D	0.866	9.842	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.032
+1.40D	0.778	9.868	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.029
+1.40D	0.694	9.894	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.026
+1.40D	0.616	9.920	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.023
+1.40D	0.541	9.946	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.020
+1.40D	0.472	9.971	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.018
+1.40D	0.407	9.997	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.015
+1.40D	0.347	10.023	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.013
+1.40D	0.292	10.049	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.011

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: SOG Under HAC Columns.ec6

DESCRIPTION: Option to remove the existing slab on grade below the HAC columns and replace with a new thickened

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.40D	0.241	10.075	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.009
+1.40D	0.196	10.101	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.007
+1.40D	0.155	10.126	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.006
+1.40D	0.118	10.152	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.004
+1.40D	0.087	10.178	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.003
+1.40D	0.061	10.204	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.002
+1.40D	0.039	10.230	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.001
+1.40D	0.022	10.256	Bottom	1.037	ACI 7.6.1.1	1.240	26.769	0.001
+1.40D	0.000	10.281	0	0.000	0	0.000	0.000	0.000
+1.40D	0.000	10.307	0	0.000	0	0.000	0.000	0.000
+1.40D	0.000	10.333	0	0.000	0	0.000	0.000	0.000

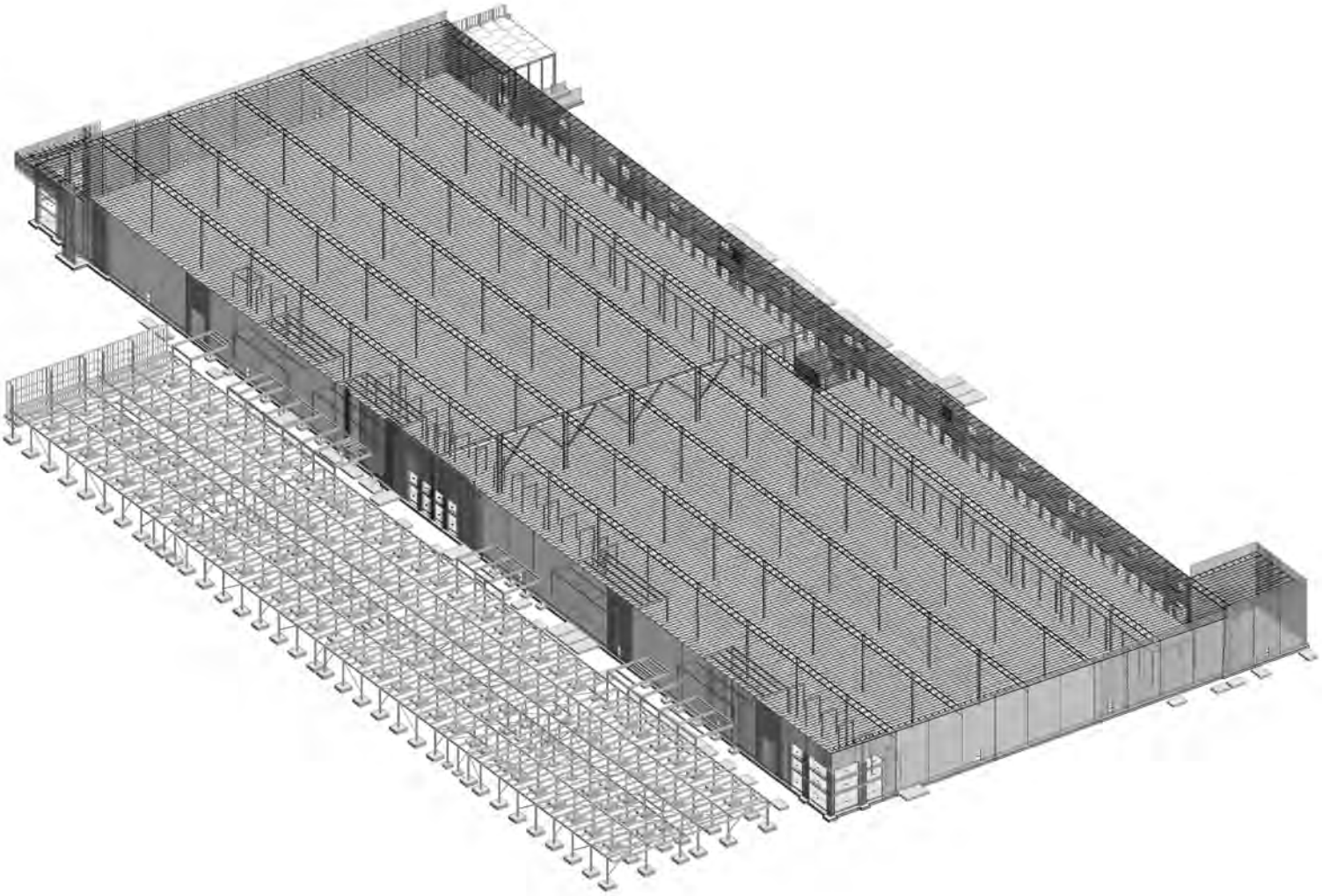
One Way Shear

Load Combination...	Left of Column #1			Right of Column #1			Left of Column #2			Right of Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	8.6	49.6	0.173	25.9	45.5	0.570	25.9	49.6	0.522	38.3	49.6	0.772
+1.20D	7.4	49.6	0.148	22.2	45.5	0.489	22.2	49.6	0.447	32.8	49.6	0.661
+0.90D	5.5	49.6	0.111	16.7	45.5	0.367	16.6	49.6	0.335	24.6	49.6	0.496
+1.20D+E	14.0	49.6	0.283	27.8	49.6	0.560	15.6	49.6	0.314	27.4	49.6	0.552
+0.90D+E	12.2	49.6	0.246	22.3	49.6	0.448	10.0	45.5	0.221	19.2	49.6	0.387

Two Way Shear

Load Combination...	Column #1			Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	72.4	164.3	0.441	70.4	164.3	0.429
+1.20D	62.1	164.3	0.378	60.4	164.3	0.367
+0.90D	46.6	164.3	0.283	45.3	164.3	0.276
+1.20D+E	89.0	164.3	0.541	88.8	164.3	0.540
+0.90D+E	73.4	164.3	0.447	73.7	164.3	0.448

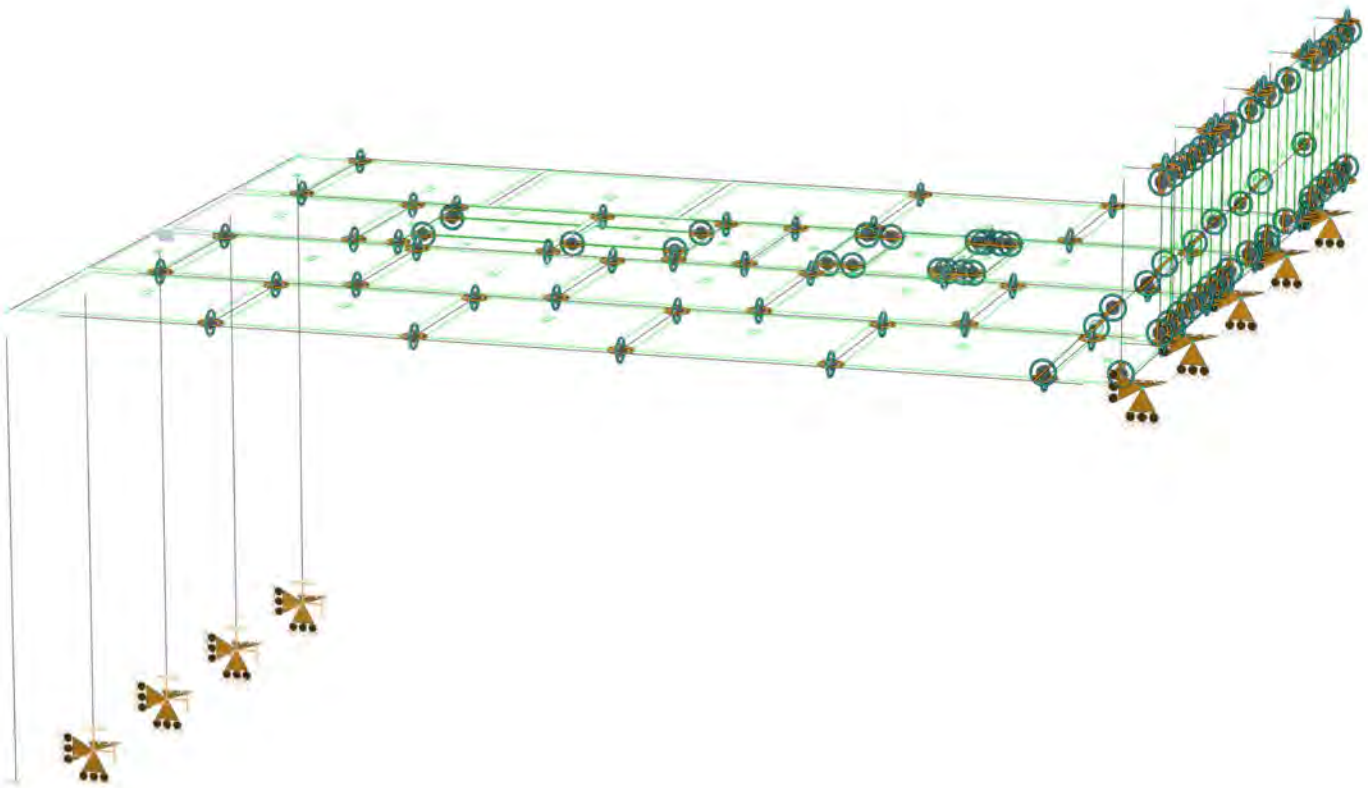
Chapter 5 - Existing Open Roof Structure Retrofit



Existing Open Roof Structure with New Mechanical Equipment and New Screenwall

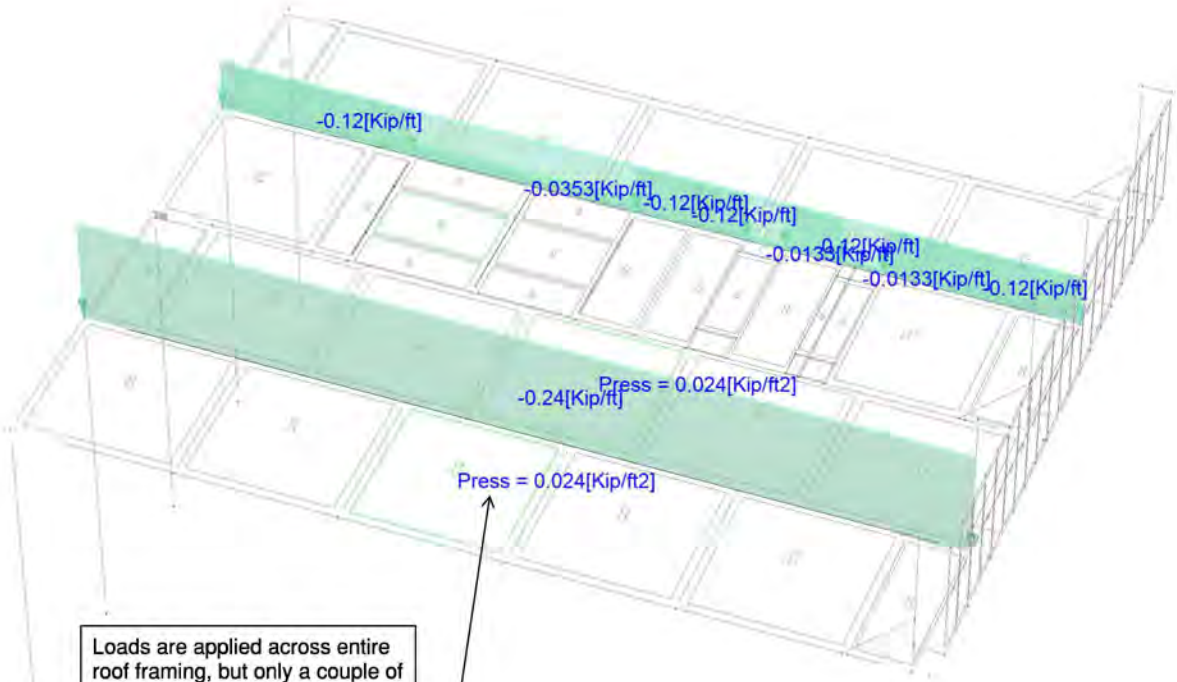


Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Supports and Hinges



Loads
Distributed area loads - Members

Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Dead Loads

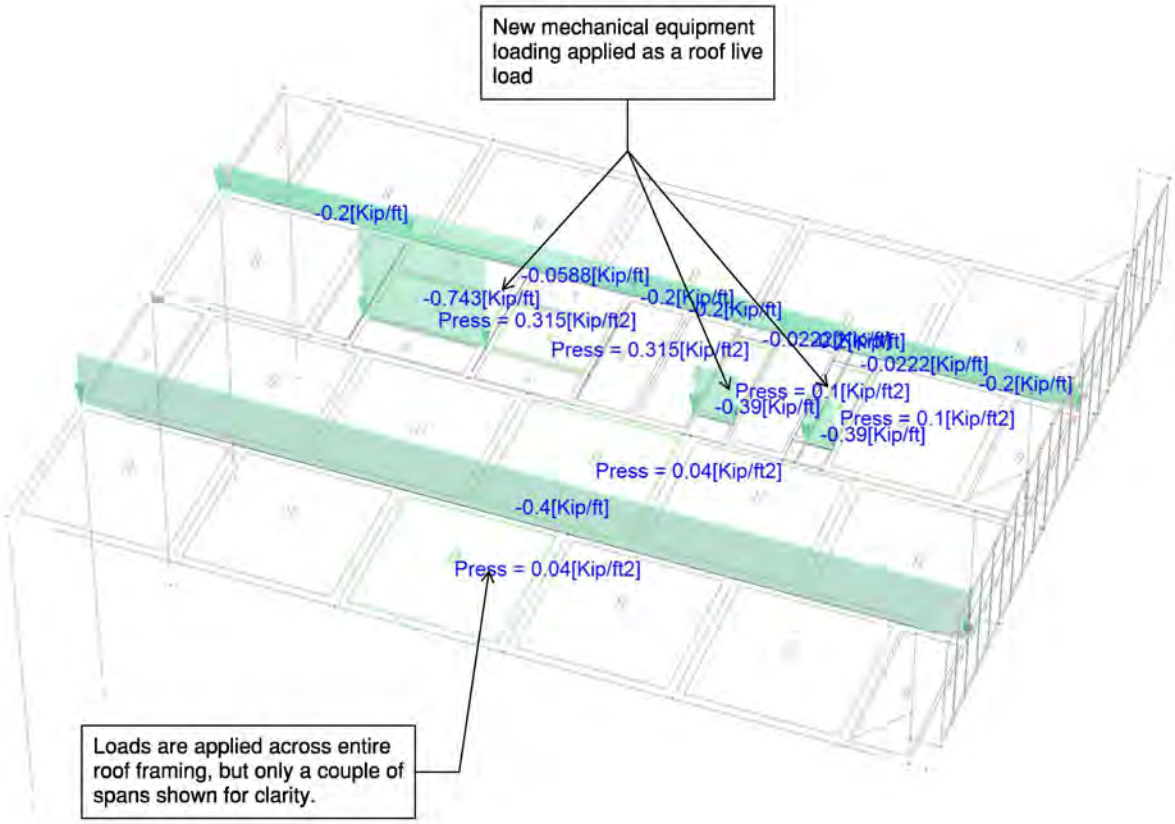


Loads are applied across entire roof framing, but only a couple of spans shown for clarity.

Load is self weight of bar grating

Loads
Distributed area loads - Members

Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Roof Live Loads



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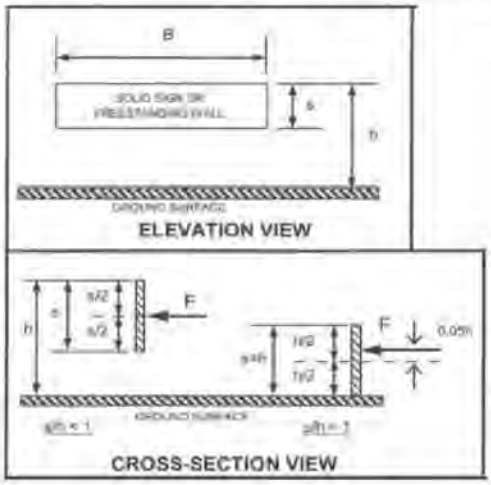
4700 LINCOLN ROAD NE SUITE 102 ALBUQUERQUE, NM 87109
PHONE: (505) 344-4080 FAX: (505) 343-8759

SHEET NO.	1	OF	1
PROJECT	Auburndale Data Center		
SUBJECT	Existing Pickleball Court Wall Wind Loads		
CLIENT	Nuclei	JOB NO.	
CALCULATED BY	PD	DATE	5/19/2026
CHECKED BY		DATE	

Existing Pickleball Court Wall Wind Loads

General Information

Height above Ground =	0 ft
Wall Height, s =	27.3 ft
Wall Length, B =	47.63 ft
Wall Length of Return, L_r =	0 ft
Building Risk Category =	II
Exposure =	C



Velocity Pressure

			Reference
Velocity Pressure Exposure, K_z =	0.96	(at the top of the wall)	(ASCE7-22, Table 26.10-1)
Topographic Factor, K_{zt} =	1.0		(ASCE7-22, Section 26.8.2)
Directionality Factor, K_d =	0.85		(ASCE7-22, Table 26.6-1)
Ground Elevation Factor, K_e =	1		(ASCE7-22, Table 26.9-1)
Basic Wind Speed, V =	140	mph	(ASCE7-22, Section 26.5)
$q_h = 0.00256K_zK_{zt}K_eV^2 =$	48.17	psf	(ASCE7-22, Eq 26.10-1)

Wind Load Force

Height of Wall From Ground, h =	27.3 ft			
Aspect Ratio, B/s =	1.74			
Clearance Ratio s/h =	1.00	=	1.00	F acts a distance above the geometric center of the sign equal to 0.05 times s
Return Corner Ratio, L_r/s =	0.00			
Return Corner Reduction =	0			(ASCE7-22, Fig 29.3-1)
s/h Reduction = $1.8 - s/h =$	0.8	(Only applies to case C)		(ASCE7-22, Fig 29.3-1)
Gust Effect Factor, G =	0.85			(ASCE7-22, Section 26.11)
Gross Area of Wall, A_s =	1301.75	ft^2		

Net Force Coefficient, C_f =	1.42	(Case A and B)	(ASCE7-22, Fig 29.3-1)
Net Force Coefficient, C_f =	2.25	(Case C, Region 0 to s)	(ASCE7-22, Fig 29.3-1)
	1.50	(Case C, Region s to 2s)	
	0.00	(Case C, Region 2s to 3s)	Non-Return Side of Wall
	0.00	(Case C, Region 3s to 4s)	
	0.00	(Case C, Region 4s to 5s)	
	0.00	(Case C, Region 5s to 10s)	
	0.00	(Case C, Region 10s and Greater)	

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consulting engineers, inc.
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SHEET NO. 2 OF 2
PROJECT Auburndale Data Center
SUBJECT Existing Pickleball Court Wall Wind Loads
CLIENT Nuclei JOB NO. 0
CALCULATED BY PD DATE 5/19/2026
CHECKED BY DATE

Existing Pickleball Court Wall Wind Loads

Reference

Wind Load Force, F = $q_h K_d G C_f A_s$ = (ASCE7-22, Eq 29.3-1)

Case	F, lb	Pressure, psf	Area of Wall, A_s	Vertical Location along Wall, ft	Horizontal Location along Wall, ft	
A	64331.11	49.42	1301.75	15.03333333	23.81	
B	64331.11	49.42	1301.75	15.03333333	14.29	
C	58502.29	78.30	747.1111	15.03333333	13.66666667	Measured From the Right Side of the Wall to the Left Side of the Wall
C	39001.52	52.20	747.1111	15.03333333	41	

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PHONE: (505) 344-4080 FAX: (505) 343-8759

SHEET NO. 1 OF 1
PROJECT Auburndale Data Center
SUBJECT MEP Equipment Wind Loads
CLIENT Nuclei JOB NO.
CALCULATED BY PD DATE 5/19/2026
CHECKED BY DATE

MEP Equipment Wind Loads

General Information

Equipment Height, h =	16.9	ft
Equipment Length, B =	49.17	ft
Equipment Width, D =	13	ft
Building Risk Category =	II	
Exposure =	C	

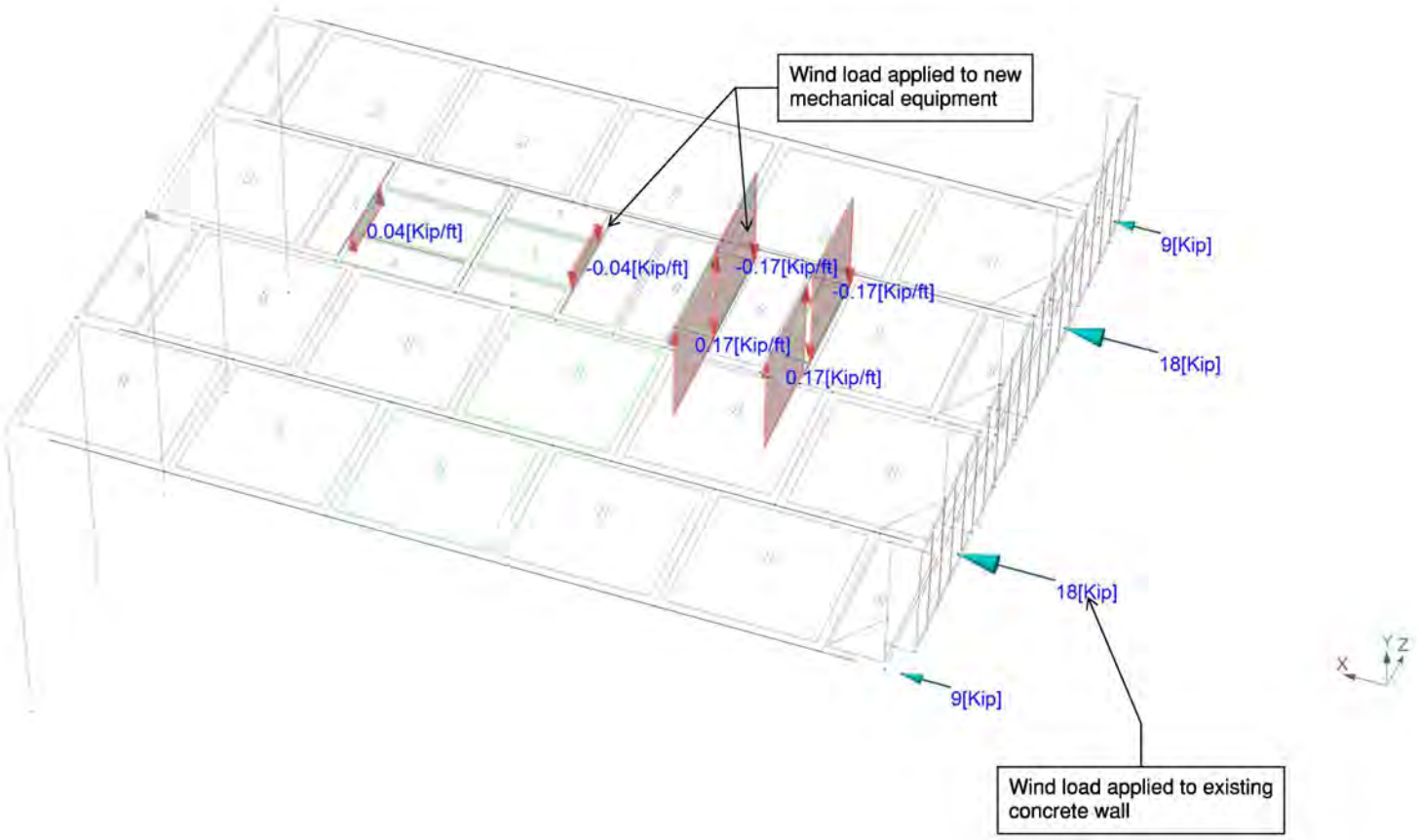
Velocity Pressure

			Reference
Velocity Pressure Exposure, K_z =	0.85		(ASCE7-22, Table 26.10-1)
Topographic Factor, K_{zt} =	1.0		(ASCE7-22, Section 26.8.2)
Directionality Factor, K_d =	0.85		(ASCE7-22, Table 26.6-1)
Ground Elevation Factor, K_e =	1		(ASCE7-22, Table 26.9-1)
Basic Wind Speed, V =	140	mph	(ASCE7-22, Section 26.5)
$q_h = 0.00256K_zK_{zt}K_eV^2 =$	42.65	psf	(ASCE7-22, Eq 26.10-1)

Wind Load Force

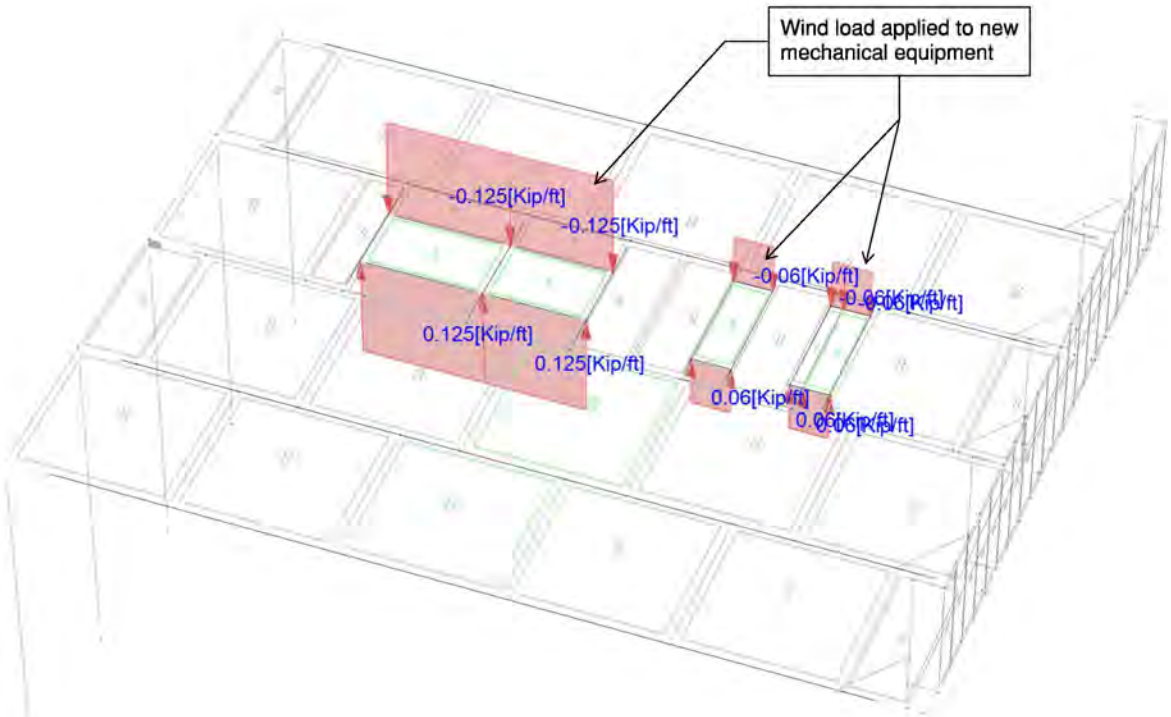
Height of Equip From Ground, h =	16.9	ft	
Aspect Ratio, h/D =	0.77		
Gust Effect Factor, G =	0.85		(ASCE7-22, Section 26.11)
Gross Area of Equipment, A_s =	831.74	ft^2	
Net Force Coefficient, C_f =	1.30		(ASCE7-22, Fig 29.4-1)
Wind Load Force, F = $q_hK_dGC_fA_s =$	33318.21	lbs	(ASCE7-22, Eq 29.4-1)
	40.06	psf	

Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Wind Loads in X Direction

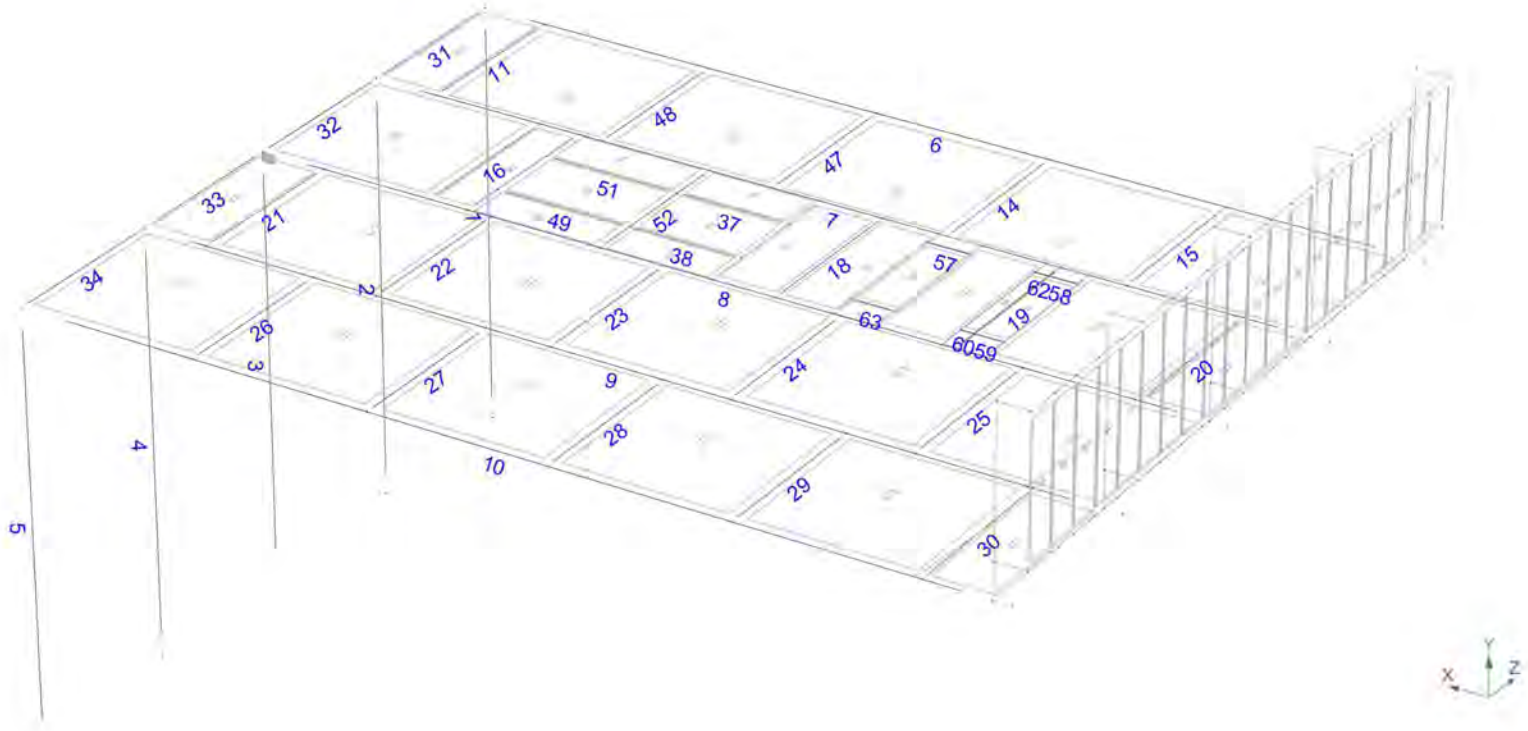


Loads
Distributed user loads - Members

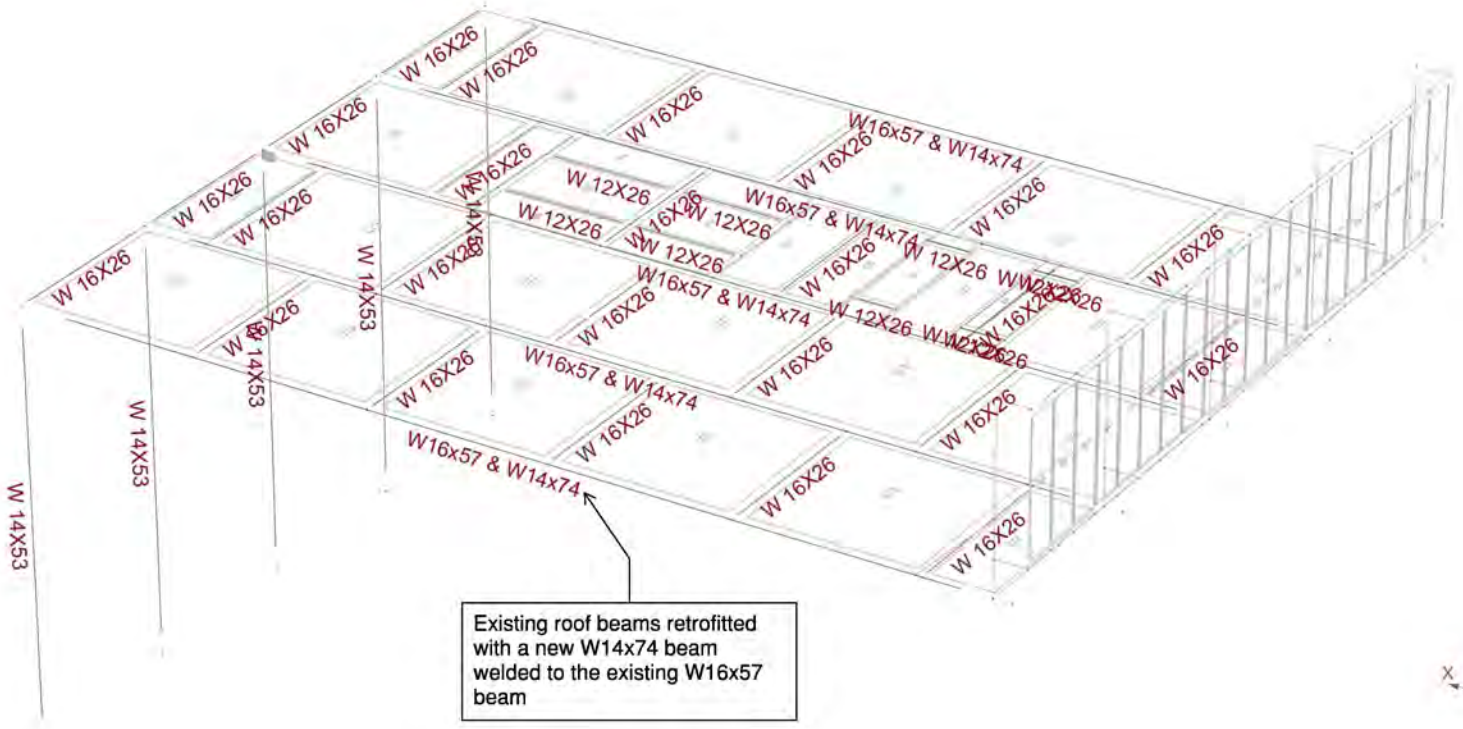
Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Wind Loads in Z Direction



Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Member Numbers



Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Member Sizes



RAM Elements Salas O'Brien

Current Date: 5/28/2026 11:58 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Existing Pickleball Court Bldg 1 and 2\Existing Pickleball Court_Rev 2 with wind at a diagonal to screenwall.retx

Steel Code Check

Summary - Group by description

Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Member Design Summary

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5Lr
D3=1.2DL+1.6Lr
D4=1.2DL+0.5W1
D5=1.2DL+0.5W2
D6=1.2DL+0.5W3
D7=1.2DL+1.6Lr+0.5W1
D8=1.2DL+1.6Lr+0.5W2
D9=1.2DL+1.6Lr+0.5W3
D10=1.2DL+W1
D11=1.2DL+W2
D12=1.2DL+W3
D13=1.2DL+W1+0.5Lr
D14=1.2DL+W2+0.5Lr
D15=1.2DL+W3+0.5Lr
D16=0.9DL+W1
D17=0.9DL+W2
D18=0.9DL+W3

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Existing W14x53 Colu...	*W 14X53*	3	D7 at 100.00%	0.55	OK	
Existing W16x26 Beam	*W 16X26*	52	D8 at 26.88%	0.15	OK	
Existing W16x26 Mom...		32	D9 at 81.25%	0.01	OK	
Existing W16x57 Girder	*W16x57 & W14x74*	8	D7 at 100.00%	**0.85**	**OK**	
New Mechanical Sup...	*W 12X26*	51	D9 at 25.00%	0.16	OK	

RAM Elements **Salas O'Brien**

Current Date: 5/29/2026 12:02 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Existing Pickleball Court Bldg 1 and 2\Existing Pickleball Court_Rev 2 with wind at a diagonal to screenwall.retx

Steel Code Check
Comprehensive Report

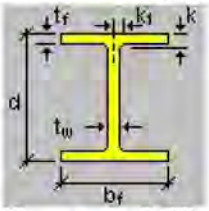
AISC 360-2016 LRFD (Hot-rolled)

Member : 3 (Existing W14x53 Column) - OK

Section information

Section name: W 14X53 **(US)**

Dimensions



bf	=	8.060	[in]	Width
d	=	13.900	[in]	Depth
k	=	1.250	[in]	Distance k
k1	=	1.000	[in]	Distance k1
tf	=	0.660	[in]	Flange thickness
tw	=	0.370	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	15.600	
Moment of Inertia (local axes) (I)	[in4]	541.000	57.700
Moment of Inertia (principal axes) (I')	[in4]	541.000	57.700
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.889	1.923
Radius of gyration (principal axes) (r')	[in]	5.889	1.923
Saint-Venant torsion constant. (J)	[in4]	1.940	
Section warping constant. (Cw)	[in6]	2540.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	77.800	14.300
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	77.800	14.300
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	77.800	14.300
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	77.800	14.300
Plastic section modulus (local axis) (Z)	[in3]	87.100	22.000
Plastic section modulus (principal axis) (Z')	[in3]	87.100	22.000
Polar radius of gyration. (ro)	[in]	6.195	
Area for shear (Aw)	[in2]	10.640	5.140
Torsional constant. (C)	[in3]	2.679	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	24.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
24.00	24.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
24.00	24.00	24.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	702.00 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	702.00	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	780.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.08			
Capacity	:	589.37 [Kip]	Reference	:	Cl.E3
Demand	:	45.12 [Kip]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.11	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.81	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	589.37	Cl.E3
Effective length (KL_{33})	[ft]	24.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	48.91	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	119.67	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	15.60	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	41.98	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	654.86	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.29			
Capacity	:	157.16 [Kip]	Reference	:	Cl.E3
Demand	:	45.12 [Kip]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.11	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.81	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	157.16	Cl.E3
Effective length (KL_{22})	[ft]	24.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	149.75	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	12.76	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	15.60	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	11.19	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	174.62	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	464.90	Cl.E4
Effective length (KL_{11})	[ft]	24.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	50.78	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	15.60	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	33.11	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	516.56	Eq.E4-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.51			
Capacity	:	326.63 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	168.09 [Kip*ft]	Ctrl Eq.	:	D7 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.11	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.81	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	326.63	Cl.F2.1
Yielding (M_n)	[Kip*ft]	362.92	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	326.63	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	6.79	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	2.22	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	22.24	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	2.17	Eq.F1-1
Critical stress (F_{cr})	[Kip/in2]	68.76	Eq.F2-4
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	362.92	Eq.F2-3

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	82.50 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.08 [Kip*ft]	Ctrl Eq.	:	D9 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.11	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.81	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	82.50	Cl.F6.1
Yielding (M_n)	[Kip*ft]	91.67	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	287.26 [Kip]	Reference	:	Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	287.26	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	319.18	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.07			
Capacity	:	154.20 [Kip]	Reference	:	Cl.G1
Demand	:	10.43 [Kip]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	154.20	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	154.20	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.55			
Ctrl Eq.	:	D7 at 100.00%	Reference	:	Eq.H1-3b

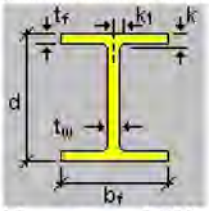
Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending:	--	0.55	Eq.H1-3a(H1-1b)
In-plane available flexural strength (M_{c33})	[Kip*ft]	326.63	Cl.H1.3
In-plane available axial compressive strength (P_c)	[Kip]	589.37	Cl.H1.3
Interaction for doubly symmetric members for out-of-plane bending:	--	0.55	Eq.H1-3b
Out-of-plane available flexural-torsional strength (M_{c33})	[Kip*ft]	184.73	Cl.H1.3
Out-of-plane available axial compressive strength (P_{co})	[Kip]	157.16	Cl.H1.3

Member : 52 (Existing W16x26 Beam) - OK

Section information

Section name: W 16X26 **(US)**

Dimensions



bf	=	5.500	[in]	Width
d	=	15.700	[in]	Depth
k	=	0.747	[in]	Distance k
k1	=	0.750	[in]	Distance k1
tf	=	0.345	[in]	Flange thickness
tw	=	0.250	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.680	
Moment of Inertia (local axes) (I)	[in4]	301.000	9.590
Moment of Inertia (principal axes) (I')	[in4]	301.000	9.590
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	6.260	1.117
Radius of gyration (principal axes) (r')	[in]	6.260	1.117
Saint-Venant torsion constant. (J)	[in4]	0.262	
Section warping constant. (Cw)	[in6]	565.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	38.400	3.490
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	38.400	3.490
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	38.400	3.490
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	38.400	3.490
Plastic section modulus (local axis) (Z)	[in3]	44.200	5.480
Plastic section modulus (principal axis) (Z')	[in3]	44.200	5.480
Polar radius of gyration. (ro)	[in]	6.359	
Area for shear (Aw)	[in2]	3.800	3.930
Torsional constant. (C)	[in3]	0.668	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design

Axial tension

Ratio	:	0.00		
Capacity	:	345.60 [Kip]	Reference	: Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	: D9 at 26.88%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	345.60	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	384.00	Eq.D2-1

Axial Compression Design

Compression in the major axis 33

Ratio	:	0.00		
Capacity	:	291.47 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D17 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	7.97	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	56.82	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	291.47	Cl.E3
Effective length (KL_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	19.17	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	779.01	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.65	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	48.67	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	323.86	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00		
Capacity	:	146.92 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D17 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	7.97	
Unstiffened element limiting slenderness (λ_r)	–	13.49	Table.B4.1a.Case1
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	56.82	
Stiffened element limiting slenderness (λ_r)	–	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	146.92	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	107.39	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	24.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	21.52	Eq.E3-2

Nominal flexural buckling strength (P_{n22})	[Kip]	163.24	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	202.66	Cl.E4
Effective length (KL_{11})	[ft]	10.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	45.57	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	7.13	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	31.59	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	225.18	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	: 0.15		
Capacity	: 133.18 [Kip*ft]	Reference	: Cl.F2.2
Demand	: 20.44 [Kip*ft]	Ctrl Eq.	: D8 at 26.88%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.97	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	56.82	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	165.75	Cl.F2.1
Yielding (M_n)	[Kip*ft]	184.17	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	133.18	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	3.95	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.38	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	11.21	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.19	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	147.98	Eq.F2-2

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 20.55 [Kip*ft]	Reference	: Cl.F6.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D9 at 26.87%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.97	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	56.82	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	20.55	Cl.F6.1
Yielding (M_n)	[Kip*ft]	22.83	Eq.F6-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	102.47 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	102.47	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	113.85	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.07		
Capacity	:	106.11 [Kip]	Reference	: Cl.G1
Demand	:	7.65 [Kip]	Ctrl Eq.	: D8 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	106.11	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	117.90	Eq.G2-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.15		
Ctrl Eq.	:	D8 at 26.88%	Reference	: Eq.H1-1b

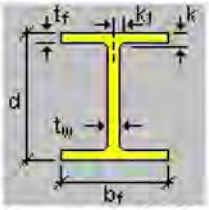
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.15	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	133.18	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	20.55	Cl.H1.1
Available axial strength (P_c)	[Kip]	345.60	Cl.H1.1

Member : 32 (Existing W16x26 Moment Frame Beam) - OK

Section information

Section name: W 16X26 **(US)**

Dimensions



bf	=	5.500	[in]	Width
d	=	15.700	[in]	Depth
k	=	0.747	[in]	Distance k
k1	=	0.750	[in]	Distance k1
tf	=	0.345	[in]	Flange thickness
tw	=	0.250	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.680	
Moment of Inertia (local axes) (I)	[in4]	301.000	9.590
Moment of Inertia (principal axes) (I')	[in4]	301.000	9.590
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	6.260	1.117
Radius of gyration (principal axes) (r')	[in]	6.260	1.117
Saint-Venant torsion constant. (J)	[in4]	0.262	
Section warping constant. (Cw)	[in6]	565.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	38.400	3.490
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	38.400	3.490
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	38.400	3.490
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	38.400	3.490
Plastic section modulus (local axis) (Z)	[in3]	44.200	5.480
Plastic section modulus (principal axis) (Z')	[in3]	44.200	5.480
Polar radius of gyration. (ro)	[in]	6.359	
Area for shear (Aw)	[in2]	3.800	3.930
Torsional constant. (C)	[in3]	0.668	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 345.60 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	345.60	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	384.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 291.47 [Kip]	Reference	: Cl.E3	
Demand	: 0.03 [Kip]	Ctrl Eq.	: D9 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.97	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	56.82	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	291.47	Cl.E3
Effective length (KL_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	19.17	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	779.01	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.65	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	48.67	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	323.86	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	146.92 [Kip]	Reference	:	Cl.E3
Demand	:	0.03 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	7.97	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	56.82	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	146.92	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	107.39	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	24.82	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.59	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	21.52	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	163.24	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	202.66	Cl.E4
Effective length (KL_{11})	[ft]	10.00	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	45.57	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	7.13	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	31.59	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	225.18	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.01			
Capacity	:	134.05 [Kip*ft]	Reference	:	Cl.F2.2
Demand	:	0.89 [Kip*ft]	Ctrl Eq.	:	D9 at 81.25%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.97	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	56.82	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	165.75	Cl.F2.1
Yielding (M_n)	[Kip*ft]	184.17	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	134.05	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	3.95	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.38	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	11.21	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.20	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	148.94	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	20.55 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D9 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	7.97	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	56.82	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	20.55	Cl.F6.1
Yielding (M_n)	[Kip*ft]	22.83	Eq.F6-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	102.47 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	102.47	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	113.85	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	106.11 [Kip]	Reference	:	Cl.G1
Demand	:	0.25 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	106.11	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	117.90	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.01			
Ctrl Eq.	:	D9 at 81.25%	Reference	:	Eq.H1-3a(H1-1b)

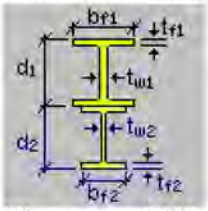
Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending:	--	0.01	Eq.H1-3a(H1-1b)
In-plane available flexural strength (M_{c33})	[Kip*ft]	165.75	Cl.H1.3
In-plane available axial compressive strength (P_c)	[Kip]	291.47	Cl.H1.3
Interaction for doubly symmetric members for out-of-plane bending:	--	0.00	Eq.H1-3b
Out-of-plane available flexural-torsional strength (M_{c33})	[Kip*ft]	111.59	Cl.H1.3
Out-of-plane available axial compressive strength (P_{co})	[Kip]	146.92	Cl.H1.3

Member : 8 (Existing W16x57 Girder) - OK

Section information

Section name: W16x57 & W14x74 **(US)**

Dimensions



bf1	=	7.125	[in]	Flange width (sup)
bf2	=	10.125	[in]	Flange width (inf)
d1	=	16.375	[in]	Depth (sup)
d2	=	14.125	[in]	Depth (inf)
tf1	=	0.688	[in]	Flange thickness (sup)
tf2	=	0.813	[in]	Flange thickness (inf)
tw1	=	0.437	[in]	Web thickness (sup)
tw2	=	0.437	[in]	Web thickness (inf)

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	38.937	
Moment of Inertia (local axes) (I)	[in4]	3776.510	182.206
Moment of Inertia (principal axes) (I')	[in4]	3776.510	182.206
Bending constant for moments (principal axis) (J')	[in]	4.085	0.000
Radius of gyration (local axes) (r)	[in]	9.848	2.163
Radius of gyration (principal axes) (r')	[in]	9.848	2.163
Saint-Venant torsion constant. (J)	[in4]	5.974	
Section warping constant. (Cw)	[in6]	16221.202	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	-3.049
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	223.297	35.991
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	277.940	35.991
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	223.297	35.991
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	277.940	35.991
Plastic section modulus (local axis) (Z)	[in3]	292.181	59.098
Plastic section modulus (principal axis) (Z')	[in3]	292.181	59.098
Polar radius of gyration. (ro)	[in]	10.534	
Area for shear (Aw)	[in2]	26.250	12.687
Torsional constant. (C)	[in3]	7.352	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	54.67

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	54.67
10.00	
10.00	
10.00	
10.00	
3.67	
1.00	

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
54.67	10.00	54.67	1.0	1.0	1.0
	10.00			1.0	
	10.00			1.0	
	10.00			1.0	
	10.00			1.0	
	3.67			1.0	
	1.00			1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	1752.19 [Kip]	Reference	:	Cl.D2
Demand	:	4.16 [Kip]	Ctrl Eq.	:	D16 at 1.83%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	1752.19	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	1946.88	Eq.D2-1

Axial Compression Design

Compression in the major axis 33

Ratio	: 0.01			
Capacity	: 1266.74 [Kip]	Reference	: Cl.E3	
Demand	: 14.56 [Kip]	Ctrl Eq.	: D8 at 8.54%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	6.23	
Unstiffened element limiting slenderness (λ_r)	–	10.80	Table.B4.1a.Case2
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	35.86	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	1266.74	Cl.E3
Effective length (KL_{33})	[ft]	54.67	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	66.61	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	64.51	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	38.94	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	36.15	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	1407.49	Eq.E7-1

Compression in the minor axis 22

Ratio	: 0.03			
Capacity	: 540.13 [Kip]	Reference	: Cl.E4	
Demand	: 14.56 [Kip]	Ctrl Eq.	: D8 at 8.54%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	6.23	
Unstiffened element limiting slenderness (λ_r)	–	10.80	Table.B4.1a.Case2
Stiffened element classification	–	Slender	
Stiffened element slenderness (λ)	–	35.86	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	1399.15	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	55.47	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	93.01	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	38.94	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	39.93	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	1554.61	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	540.13	Cl.E4
Effective length (KL_{11})	[ft]	54.67	Cl.E2
Flexural constant (H)	–	0.92	Eq.E4-8
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	17.57	Eq.E4-3
Elastic torsional buckling stress (F_{ez})	[Kip/in2]	17.92	Eq.E4-7
Effective area of the cross section based on the effective width (A...	[in2]	38.94	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	15.41	Eq.E3-3

Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	600.15	Eq.E7-1

Flexural Design ✔

Bending about major axis, M33

Ratio	: 0.84		
Capacity	: 199.96 [Kip*ft]	Reference	: Cl.F12.2
Demand	: -168.09 [Kip*ft]	Ctrl Eq.	: D7 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.23	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	19.17	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	35.86	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	837.36	Cl.F12.1
Yielding (M_n)	[Kip*ft]	930.40	Eq.F12-1
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	837.36	Cl.F12.1
Yielding (M_n)	[Kip*ft]	930.40	Eq.F12-1
Factored lateral-torsional buckling strength about a geometric axis(ϕM_n):	[Kip*ft]	199.96	Cl.F12.2
Lateral-torsional buckling modification factor (C_b)	--	1.17	Eq.F1-1
Critical stress (F_{cr})	[Kip/in2]	11.94	Cl.F12
Elastic lateral-torsional buckling moment (M_{cr})	[Kip*ft]	276.54	Galambos:Eq.5.10
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	222.17	Eq.F12-1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 134.97 [Kip*ft]	Reference	: Cl.F12.1
Demand	: -0.04 [Kip*ft]	Ctrl Eq.	: D10 at 9.15%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.23	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	19.17	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	35.86	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	134.97	Cl.F12.1
Yielding (M_n)	[Kip*ft]	149.96	Eq.F12-1
Factored yielding strength(ϕM_n):	[Kip*ft]	134.97	Cl.F12.1
Yielding (M_n)	[Kip*ft]	149.96	Eq.F12-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	708.75 [Kip]	Reference	:	Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D9 at 23.81%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	708.75	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Cl.G4
Nominal shear strength (V_n)	[Kip]	787.50	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.13			
Capacity	:	342.56 [Kip]	Reference	:	Cl.G1
Demand	:	43.24 [Kip]	Ctrl Eq.	:	D7 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	342.56	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Cl.G4
Nominal shear strength (V_n)	[Kip]	380.62	Eq.G4-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.85			
Ctrl Eq.	:	D7 at 100.00%	Reference	:	Eq.H1-1b

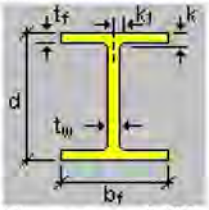
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.85	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	199.96	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	134.97	Cl.H1.1
Available axial strength (P_c)	[Kip]	550.58	Cl.H1.1

Member : 51 (New Mechanical Support Beam) - OK

Section information

Section name: W 12X26 **(US)**

Dimensions



bf	=	6.490	[in]	Width
d	=	12.200	[in]	Depth
k	=	0.680	[in]	Distance k
k1	=	0.750	[in]	Distance k1
tf	=	0.380	[in]	Flange thickness
tw	=	0.230	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	7.650	
Moment of Inertia (local axes) (I)	[in4]	204.000	17.300
Moment of Inertia (principal axes) (I')	[in4]	204.000	17.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.164	1.504
Radius of gyration (principal axes) (r')	[in]	5.164	1.504
Saint-Venant torsion constant. (J)	[in4]	0.300	
Section warping constant. (Cw)	[in6]	607.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	33.400	5.340
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	33.400	5.340
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	33.400	5.340
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	33.400	5.340
Plastic section modulus (local axis) (Z)	[in3]	37.200	8.170
Plastic section modulus (principal axis) (Z')	[in3]	37.200	8.170
Polar radius of gyration. (ro)	[in]	5.378	
Area for shear (Aw)	[in2]	4.930	2.810
Torsional constant. (C)	[in3]	0.751	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	7.71

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
7.71	7.71

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
7.71	7.71	7.71	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 344.25 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	344.25	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	382.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 317.07 [Kip]	Reference	: Cl.E3	
Demand	: 0.01 [Kip]	Ctrl Eq.	: D9 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	8.54	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	47.13	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	317.07	Cl.E3
Effective length (KL_{33})	[ft]	7.71	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	17.91	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	892.05	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.21	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	48.84	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	352.30	Eq.E7-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	253.24 [Kip]	Reference	:	Cl.E3
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	8.54	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	47.13	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	253.24	Cl.E3
Effective length (KL_{22})	[ft]	7.71	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	61.51	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	75.65	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.42	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	37.92	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	281.38	Eq.E7-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	272.07	Cl.E4
Effective length (KL_{11})	[ft]	7.71	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	106.87	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	7.35	
Critical stress for torsional or flexural-torsional buckling (F_{cr11})	[Kip/in2]	41.11	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	302.30	Eq.E7-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.16			
Capacity	:	139.50 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	22.15 [Kip*ft]	Ctrl Eq.	:	D9 at 25.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.54	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	47.13	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	139.50	Cl.F2.1
Yielding (M_n)	[Kip*ft]	155.00	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	139.50	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	5.31	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.75	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	14.89	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	1.16	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	155.00	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	30.64 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	8.54	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	47.13	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	30.64	Cl.F6.1
Yielding (M_n)	[Kip*ft]	34.04	Eq.F6-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	133.17 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	133.17	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	147.97	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.09			
Capacity	:	84.30 [Kip]	Reference	:	Cl.G1
Demand	:	7.68 [Kip]	Ctrl Eq.	:	D9 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	84.30	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	84.30	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.16			
Ctrl Eq.	:	D9 at 25.00%	Reference	:	Eq.H1-3a(H1-1b)

Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending:	--	0.16	Eq.H1-3a(H1-1b)
In-plane available flexural strength (M_{c33})	[Kip*ft]	139.50	Cl.H1.3
In-plane available axial compressive strength (P_c)	[Kip]	317.07	Cl.H1.3
Interaction for doubly symmetric members for out-of-plane bending:	--	0.02	Eq.H1-3b
Out-of-plane available flexural-torsional strength (M_{c33})	[Kip*ft]	126.54	Cl.H1.3
Out-of-plane available axial compressive strength (P_{co})	[Kip]	253.24	Cl.H1.3

Chavez-Grieves
consulting engineers, inc.

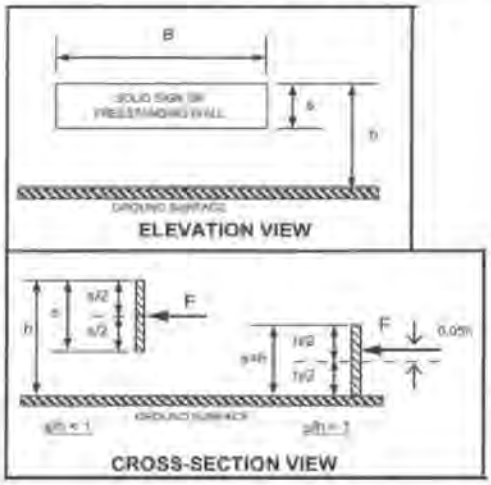
4700 LINCOLN ROAD NE SUITE 102 ALBUQUERQUE, NM 87109
PHONE: (505) 344-4080 FAX: (505) 343-8759

SHEET NO. 1 OF 1
PROJECT Auburndale Data Center
SUBJECT New Pickleball Court Screenwall Wind Loads
CLIENT Nuclei JOB NO.
CALCULATED BY PD DATE 5/19/2026
CHECKED BY DATE

New Pickleball Court Screenwall Wind Loads

General Information

Height above Ground =	28.5 ft
Wall Height, s =	8.0 ft
Wall Length, B =	50.00 ft
Wall Length of Return, L_r =	0 ft
Building Risk Category =	II
Exposure =	C



Velocity Pressure

			Reference
Velocity Pressure Exposure, K_z =	1.04	(at the top of the wall)	(ASCE7-22, Table 26.10-1)
Topographic Factor, K_{zt} =	1.0		(ASCE7-22, Section 26.8.2)
Directionality Factor, K_d =	0.85		(ASCE7-22, Table 26.6-1)
Ground Elevation Factor, K_e =	1		(ASCE7-22, Table 26.9-1)
Basic Wind Speed, V =	140	mph	(ASCE7-22, Section 26.5)
$q_h = 0.00256K_zK_{zt}K_eV^2 =$	52.18	psf	(ASCE7-22, Eq 26.10-1)

Wind Load Force

Height of Wall From Ground, h =	36.5 ft			
Aspect Ratio, B/s =	6.25			
Clearance Ratio s/h =	0.22	<	1.00	F acts through the geometric center of the sign
Return Corner Ratio, L_r/s =	0.00			
Return Corner Reduction =	0			(ASCE7-22, Fig 29.3-1)
s/h Reduction = $1.8 - s/h =$	1	(Only applies to case C)		(ASCE7-22, Fig 29.3-1)
Gust Effect Factor, G =	0.85			(ASCE7-22, Section 26.11)
Gross Area of Wall, A_s =	400.00	ft^2		

			Reference
Net Force Coefficient, C_f =	1.81	(Case A and B)	(ASCE7-22, Fig 29.3-1)
Net Force Coefficient, C_f =	3.30	(Case C, Region 0 to s)	(ASCE7-22, Fig 29.3-1)
	2.15	(Case C, Region s to 2s)	
	1.55	(Case C, Region 2s to 3s)	
	1.05	(Case C, Region 3s to 10s)	

Non-Return Side of Wall

Chavez-Grieves
consulting engineers, inc.
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SHEET NO. 2 OF 2
PROJECT Auburndale Data Center
SUBJECT New Pickleball Court Screenwall Wind Loads
CLIENT Nuclei JOB NO. 0
CALCULATED BY PD DATE 5/19/2026
CHECKED BY DATE

New Pickleball Court Screenwall Wind Loads

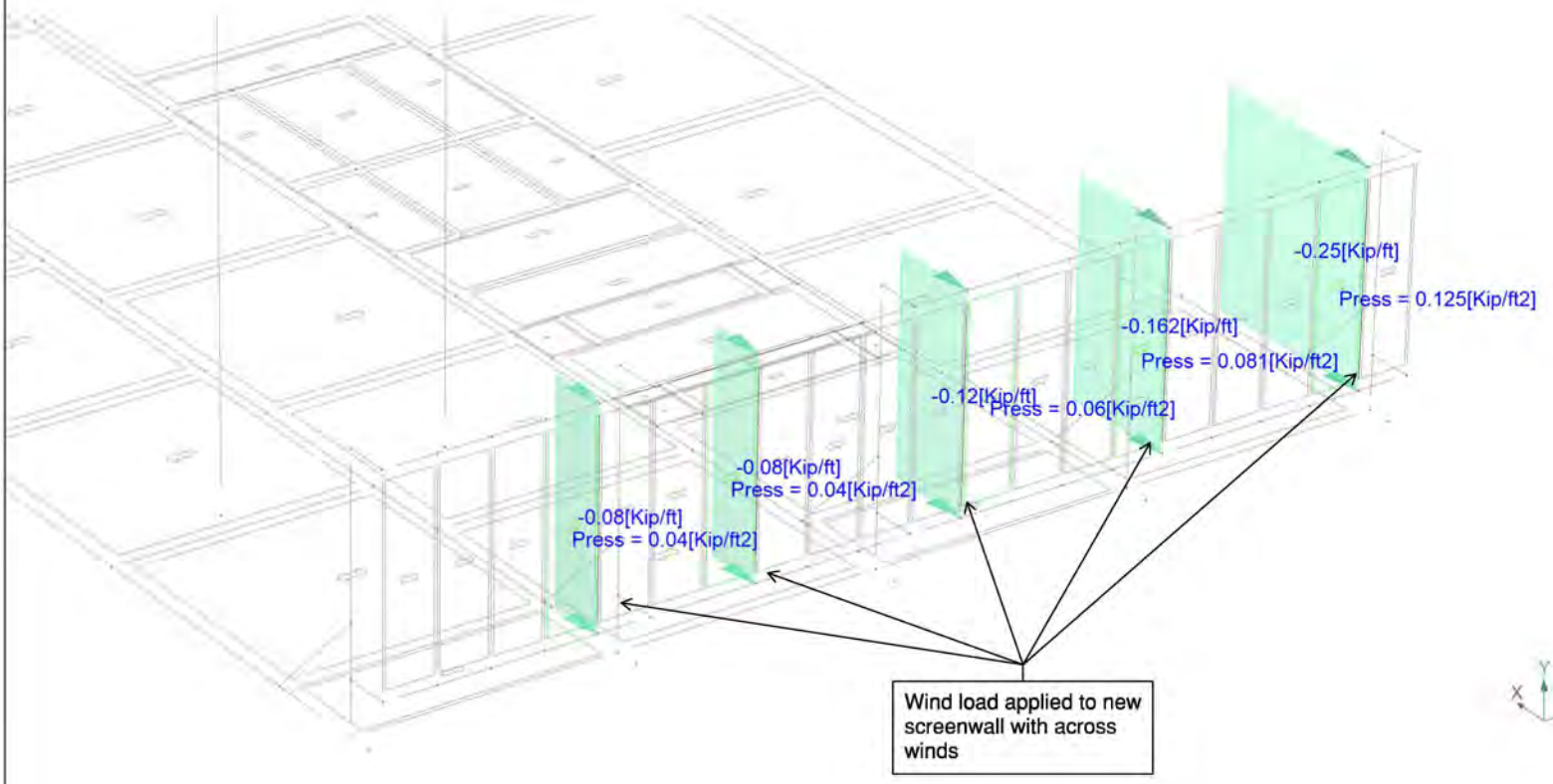
Reference

Wind Load Force, F = $q_h K_d G C_f A_s$ = (ASCE7-22, Eq 29.3-1)

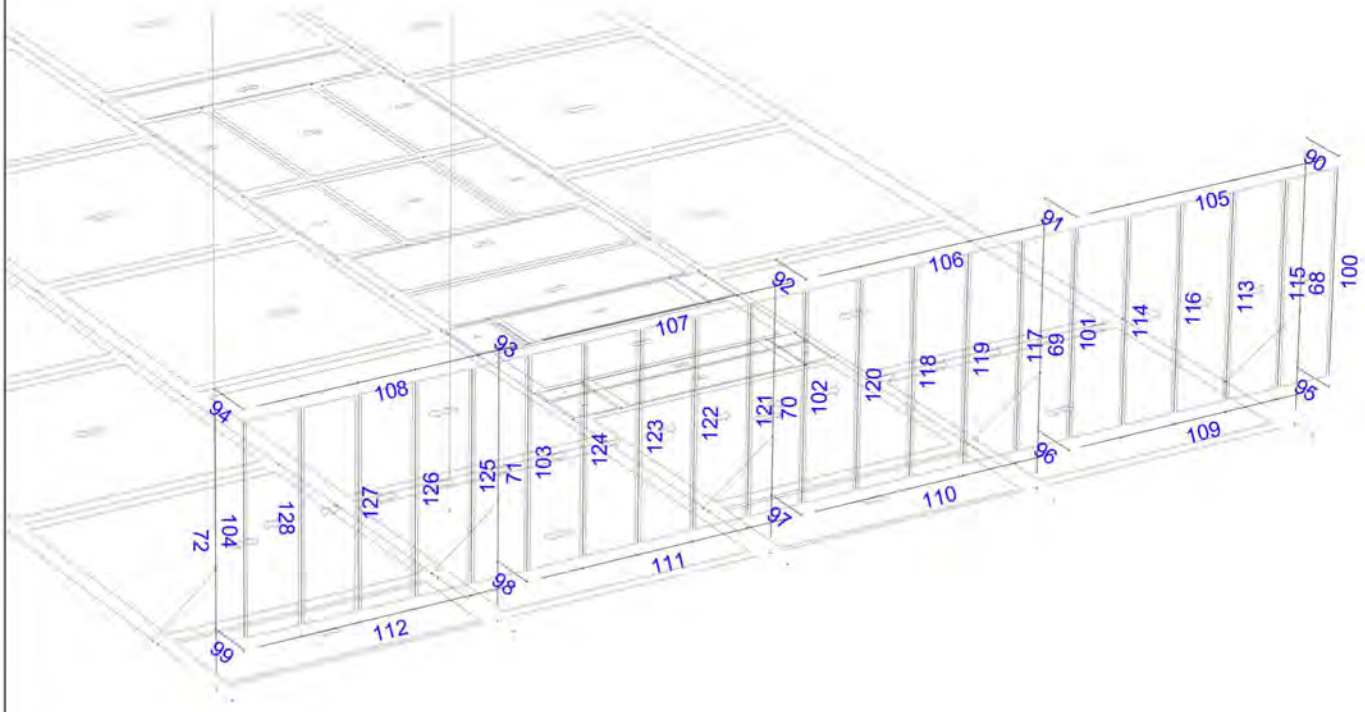
Case	F, lb	Pressure, psf	Area of Wall, A_s	Vertical Location along Wall, ft	Horizontal Location along Wall, ft	
A	27296.43	68.24	400.00	18.25	25.00	
B	27296.43	68.24	400.00	18.25	15.00	
C	7962.71	124.42	64	18.25	4	Measured From the Right Side of the Wall to the Left Side of the Wall
C	5187.83	81.06	64	18.25	12	
C	3740.06	58.44	64	18.25	20	
C	2533.59	39.59	64	18.25	28	

Loads
Distributed area loads - Members

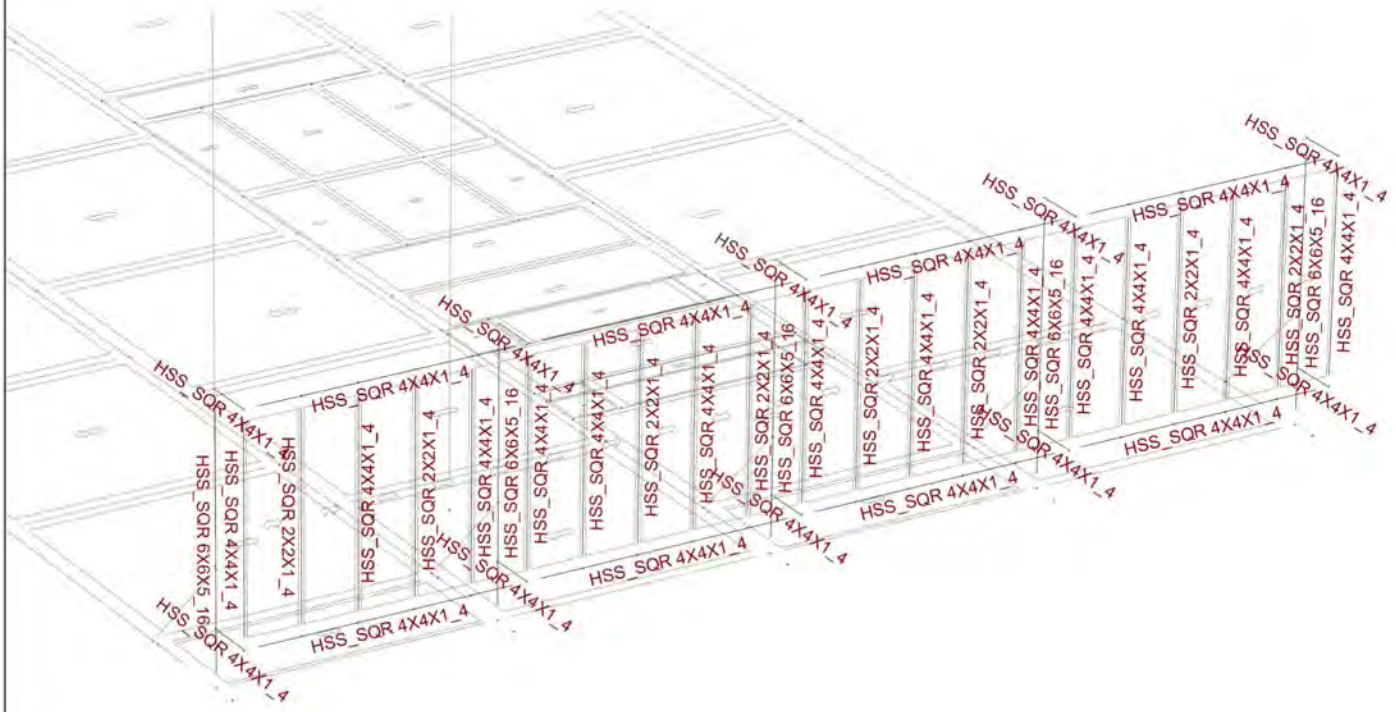
Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Wind Loads Applied to Screenwall



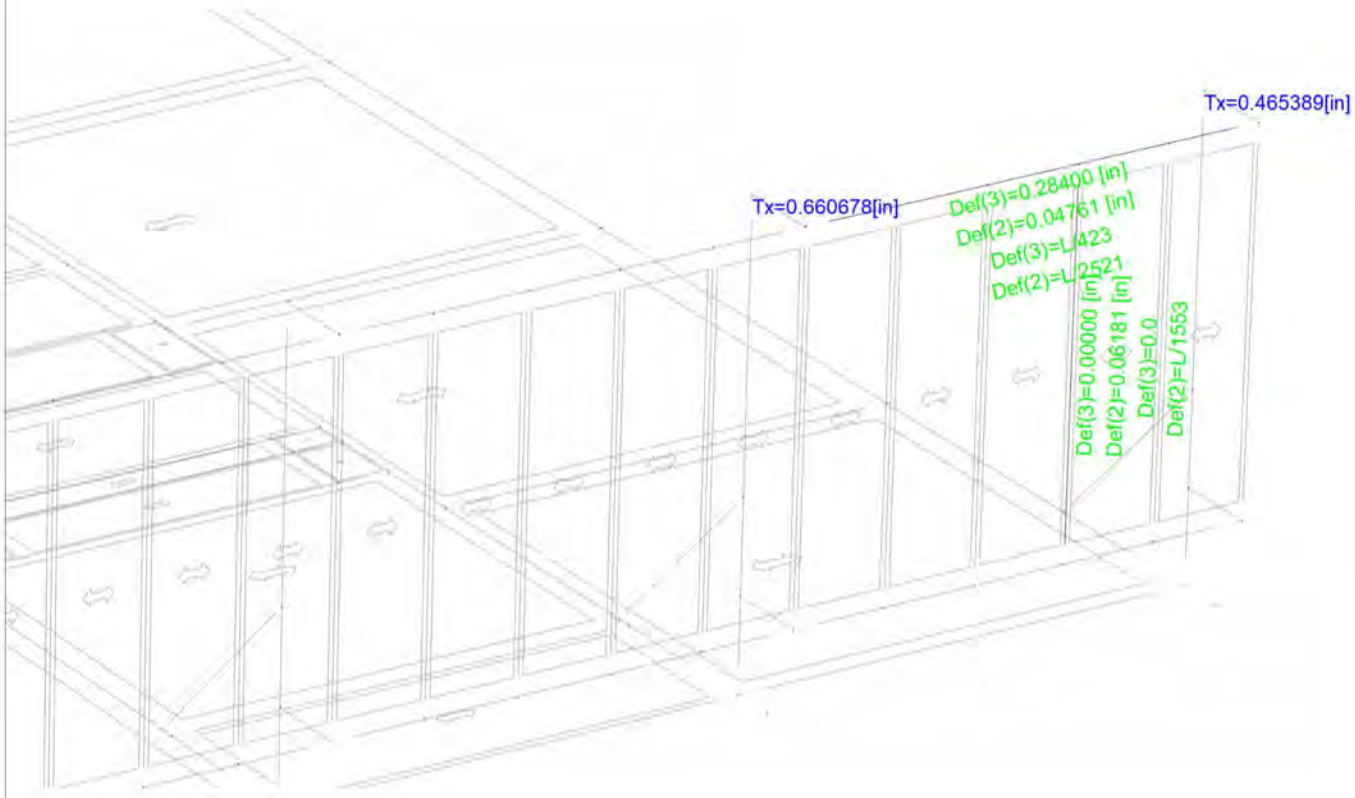
New ScreenwallMember Numbers



New Screenwall Member Sizes



New Screenwall Deflections



RAM Elements Salas O'Brien

Current Date: 5/29/2026 12:09 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Existing Pickleball Court Bldg 1 and 2\Existing Pickleball Court_Rev 2 with wind at a diagonal to screenwall.retx

Steel Code Check
Summary - Group by description

Existing Open Roof Structure with New Mechanical Equipment and New Screenwall Member Design Summary

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5Lr
D3=1.2DL+1.6Lr
D4=1.2DL+0.5W1
D5=1.2DL+0.5W2
D6=1.2DL+0.5W3
D7=1.2DL+1.6Lr+0.5W1
D8=1.2DL+1.6Lr+0.5W2
D9=1.2DL+1.6Lr+0.5W3
D10=1.2DL+W1
D11=1.2DL+W2
D12=1.2DL+W3
D13=1.2DL+W1+0.5Lr
D14=1.2DL+W2+0.5Lr
D15=1.2DL+W3+0.5Lr
D16=0.9DL+W1
D17=0.9DL+W2
D18=0.9DL+W3

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Cantilever Beam	*HSS_SQR 4X4X1_4*	**96**	D14 at 100.00%	0.52	OK	
Edge Vertical Member		**113**	D13 at 50.00%	0.11	OK	
Intermediate Vertical...	*HSS_SQR 2X2X1_4*	**115**	D11 at 50.00%	**0.56**	**OK**	
Screenwall Bottom B...	*HSS_SQR 4X4X1_4*	**109**	D13 at 40.00%	0.38	OK	
Screenwall Column	*HSS_SQR 6X6X5_16*	**69**	D14 at 40.00%	0.32	OK	
Screenwall Top Beam	*HSS_SQR 4X4X1_4*	**105**	D13 at 40.00%	0.39	OK	

RAM Elements **Salas O'Brien**

Current Date: 5/29/2026 12:11 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Existing Pickleball Court Bldg 1 and 2\Existing Pickleball Court_Rev 2 with wind at a diagonal to screenwall.retx

Steel Code Check
Comprehensive Report

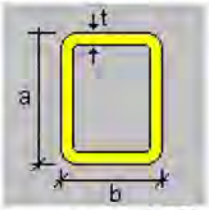
AISC 360-2016 LRFD (Hot-rolled)

Member : 96 (Cantilever Beam) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	1.75

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
1.75	1.75

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
1.75	1.75	1.75	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.03			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	3.95 [Kip]	Ctrl Eq.	:	D17 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.03			
Capacity	:	149.55 [Kip]	Reference	:	Cl.E3
Demand	:	4.22 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	149.55	Cl.E3
Effective length (KL_{33})	[ft]	1.75	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	13.80	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	1502.19	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	49.31	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	166.17	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.03			
Capacity	:	149.55 [Kip]	Reference	:	Cl.E3
Demand	:	4.22 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	149.55	Cl.E3
Effective length (KL_{22})	[ft]	1.75	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	13.80	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	1502.19	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	49.31	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	166.17	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.50			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-8.88 [Kip*ft]	Ctrl Eq.	:	D14 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	: 0.00			
Capacity	: 17.59 [Kip*ft]	Reference	: Cl.F7.1	
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D15 at 100.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	: 0.00			
Capacity	: 41.53 [Kip]	Reference	: Cl.G1	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D12 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	: 0.11			
Capacity	: 41.53 [Kip]	Reference	: Cl.G1	
Demand	: 4.57 [Kip]	Ctrl Eq.	: D14 at 100.00%	

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D12 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design

Combined flexure and axial

Ratio	:	0.52			
Ctrl Eq.	:	D14 at 100.00%	Reference	:	Eq.H1-1b

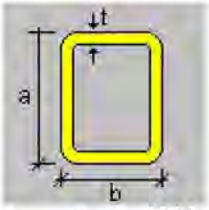
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.52	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	151.65	Cl.H1.1

Member : 113 (Edge Vertical Member) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000

Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	8.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
8.00	8.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
8.00	8.00	8.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.14 [Kip]	Ctrl Eq.	:	D11 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 113.35 [Kip]	Reference	: Cl.E3	
Demand	: 0.13 [Kip]	Ctrl Eq.	: D13 at 100.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	14.17	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	113.35	Cl.E3
Effective length (KL_{33})	[ft]	8.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	63.10	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	71.88	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	37.37	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	125.94	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.00			
Capacity	: 113.35 [Kip]	Reference	: Cl.E3	
Demand	: 0.13 [Kip]	Ctrl Eq.	: D13 at 100.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	14.17	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	14.17	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	113.35	Cl.E3
Effective length (KL_{22})	[ft]	8.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	63.10	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	71.88	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	37.37	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	125.94	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.11			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	2.00 [Kip*ft]	Ctrl Eq.	:	D11 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.02			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	1.00 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.04			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.57 [Kip*ft]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.11			
Ctrl Eq.	:	D13 at 50.00%	Reference	:	Eq.H1-1b

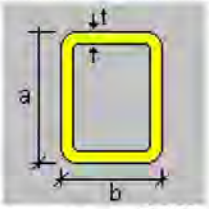
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.11	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	113.35	Cl.H1.1

Member : 115 (Intermediate Vertical Member) - OK

Section information

Section name: HSS_SQR 2X2X1_4 **(US)**

Dimensions



a	=	2.000	[in]	Height
b	=	2.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	1.510	
Moment of Inertia (local axes) (I)	[in4]	0.747	0.747
Moment of Inertia (principal axes) (I')	[in4]	0.747	0.747
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	0.703	0.703
Radius of gyration (principal axes) (r')	[in]	0.703	0.703
Saint-Venant torsion constant. (J)	[in4]	1.310	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	0.747	0.747
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	0.747	0.747
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	0.747	0.747
Plastic section modulus (local axis) (Z)	[in3]	0.964	0.964
Plastic section modulus (principal axis) (Z')	[in3]	0.964	0.964
Polar radius of gyration. (ro)	[in]	0.990	
Area for shear (Aw)	[in2]	0.606	0.606
Torsional constant. (C)	[in3]	1.405	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	8.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
8.00	8.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
8.00	8.00	8.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	67.95 [Kip]	Reference	:	Cl.D2
Demand	:	0.13 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	67.95	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	75.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	18.31 [Kip]	Reference	:	Cl.E3
Demand	:	0.14 [Kip]	Ctrl Eq.	:	D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	18.31	Cl.E3
Effective length (KL_{33})	[ft]	8.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	136.49	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	15.36	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	13.47	Eq.E3-3
Nominal flexural buckling strength (P_{n33})	[Kip]	20.35	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.01			
Capacity	:	18.31 [Kip]	Reference	:	Cl.E3
Demand	:	0.14 [Kip]	Ctrl Eq.	:	D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	5.58	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	5.58	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	18.31	Cl.E3
Effective length (KL_{22})	[ft]	8.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	136.49	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	15.36	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	1.51	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	13.47	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	20.35	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.55			
Capacity	:	3.62 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-2.00 [Kip*ft]	Ctrl Eq.	:	D10 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	3.62	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	6.45	Eq.F7-12
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	182.87	Eq.F7-13
Lateral-torsional buckling modification factor (C_b)	--	1.14	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	4.02	Eq.F7-10

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	3.62 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	5.58	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	3.62	Cl.F7.1
Yielding (M_n)	[Kip*ft]	4.02	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	16.37 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.06			
Capacity	:	16.37 [Kip]	Reference	:	Cl.G1
Demand	:	1.00 [Kip]	Ctrl Eq.	:	D13 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	16.37	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	18.19	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.02			
Capacity	:	3.16 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.07 [Kip*ft]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	3.16	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	3.51	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.56			
Ctrl Eq.	:	D11 at 50.00%	Reference	:	Eq.H1-1b

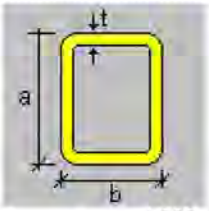
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.56	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	3.62	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	3.62	Cl.H1.1
Available axial strength (P_c)	[Kip]	18.31	Cl.H1.1

Member : 109 (Screenwall Bottom Beam) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	151.65 [Kip]	Reference	:	Cl.D2
Demand	:	0.02 [Kip]	Ctrl Eq.	:	D9 at 80.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	96.22 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	96.22	Cl.E3
Effective length (KL_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	31.73	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	106.91	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	96.22 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	96.22	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	31.73	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	106.91	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.05			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.81 [Kip*ft]	Ctrl Eq.	:	D1 at 46.67%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	

Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	
Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.34		
Capacity	:	17.59 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-5.95 [Kip*ft]	Ctrl Eq.	: D13 at 40.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	
Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.05		
Capacity	:	41.53 [Kip]	Reference	: Cl.G1
Demand	:	1.98 [Kip]	Ctrl Eq.	: D14 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.01		
Capacity	:	41.53 [Kip]	Reference	: Cl.G1
Demand	:	0.30 [Kip]	Ctrl Eq.	: D1 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.04			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.58 [Kip*ft]	Ctrl Eq.	:	D7 at 80.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.38			
Ctrl Eq.	:	D13 at 40.00%	Reference	:	Eq.H1-1b

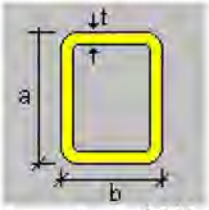
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.38	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	151.65	Cl.H1.1

Member : 69 (Screenwall Column) - OK

Section information

Section name: HSS_SQR 6X6X5_16 **(US)**

Dimensions



a	=	6.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.291	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	6.430	
Moment of Inertia (local axes) (I)	[in4]	34.300	34.300
Moment of Inertia (principal axes) (I')	[in4]	34.300	34.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.310	2.310
Radius of gyration (principal axes) (r')	[in]	2.310	2.310
Saint-Venant torsion constant. (J)	[in4]	55.400	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000

Top elastic section modulus of the section (local axis) (Ssup)	[in3]	11.400	11.400
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	11.400	11.400
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	11.400	11.400
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	11.400	11.400
Plastic section modulus (local axis) (Z)	[in3]	13.600	13.600
Plastic section modulus (principal axis) (Z')	[in3]	13.600	13.600
Polar radius of gyration. (ro)	[in]	3.266	
Area for shear (Aw)	[in2]	2.984	2.984
Torsional constant. (C)	[in3]	18.871	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.03			
Capacity	:	289.35 [Kip]	Reference	:	Cl.D2
Demand	:	9.40 [Kip]	Ctrl Eq.	:	D16 at 20.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	289.35	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	321.50	Eq.D2-1

Axial Compression Design ✓

Compression in the major axis 33

Ratio	: 0.06			
Capacity	: 237.52 [Kip]	Reference	: Cl.E3	
Demand	: 13.20 [Kip]	Ctrl Eq.	: D14 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	17.62	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	17.62	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	237.52	Cl.E3
Effective length (KL_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	–	51.96	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	106.03	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.43	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	41.04	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	263.91	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.06			
Capacity	: 237.52 [Kip]	Reference	: Cl.E3	
Demand	: 13.20 [Kip]	Ctrl Eq.	: D14 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Non slender	
Unstiffened element slenderness (λ)	–	17.62	
Unstiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Stiffened element classification	–	Non slender	
Stiffened element slenderness (λ)	–	17.62	
Stiffened element limiting slenderness (λ_r)	–	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	237.52	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	–	51.96	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	106.03	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	6.43	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	41.04	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	263.91	Eq.E3-1

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.30			
Capacity	:	51.00 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	15.48 [Kip*ft]	Ctrl Eq.	:	D14 at 40.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	17.62	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	17.62	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	51.00	Cl.F7.1
Yielding (M_n)	[Kip*ft]	56.67	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	51.00 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-0.02 [Kip*ft]	Ctrl Eq.	:	D10 at 40.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	17.62	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	17.62	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	51.00	Cl.F7.1
Yielding (M_n)	[Kip*ft]	56.67	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	80.57 [Kip]	Reference	:	Cl.G1
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D10 at 20.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	80.57	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	89.52	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.10			
Capacity	:	80.57 [Kip]	Reference	:	Cl.G1
Demand	:	7.98 [Kip]	Ctrl Eq.	:	D14 at 20.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	80.57	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	89.52	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	42.46 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	-0.02 [Kip*ft]	Ctrl Eq.	:	D10 at 20.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	42.46	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	47.18	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.32			
Ctrl Eq.	:	D14 at 40.00%	Reference	:	Eq.H1-1b

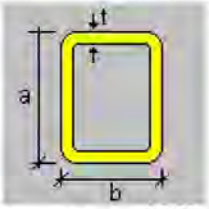
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.32	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	51.00	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	51.00	Cl.H1.1
Available axial strength (P_c)	[Kip]	237.52	Cl.H1.1

Member : 105 (Screenwall Top Beam) - OK

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.690	4.690
Plastic section modulus (principal axis) (Z')	[in3]	4.690	4.690
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 151.65 [Kip]	Reference	: Cl.D2	
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	151.65	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	168.50	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 96.22 [Kip]	Reference	: Cl.E3	
Demand	: 0.02 [Kip]	Ctrl Eq.	: D9 at 80.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	96.22	Cl.E3
Effective length (KL_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	31.73	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	106.91	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	96.22 [Kip]	Reference	:	Cl.E3
Demand	:	0.02 [Kip]	Ctrl Eq.	:	D9 at 80.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	96.22	Cl.E3
Effective length (KL_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	31.73	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	106.91	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.05			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.81 [Kip*ft]	Ctrl Eq.	:	D1 at 46.67%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	
Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.35			
Capacity	:	17.59 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-6.13 [Kip*ft]	Ctrl Eq.	:	D13 at 40.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	137.27	
Limiting slenderness for compact unstiffened element (λ_p)	--	58.28	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	33.72	

Limiting slenderness for compact stiffened element (λ_p)	--	26.97	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	17.59	Cl.F7.1
Yielding (M_n)	[Kip*ft]	19.54	Eq.F7-1

Shear Design ✓

Shear in major axis 33

Ratio	:	0.05			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	2.04 [Kip]	Ctrl Eq.	:	D13 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	41.53 [Kip]	Reference	:	Cl.G1
Demand	:	0.29 [Kip]	Ctrl Eq.	:	D1 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	41.53	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	46.15	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.04			
Capacity	:	14.77 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.55 [Kip*ft]	Ctrl Eq.	:	D7 at 40.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	14.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	16.41	Eq.H3-1

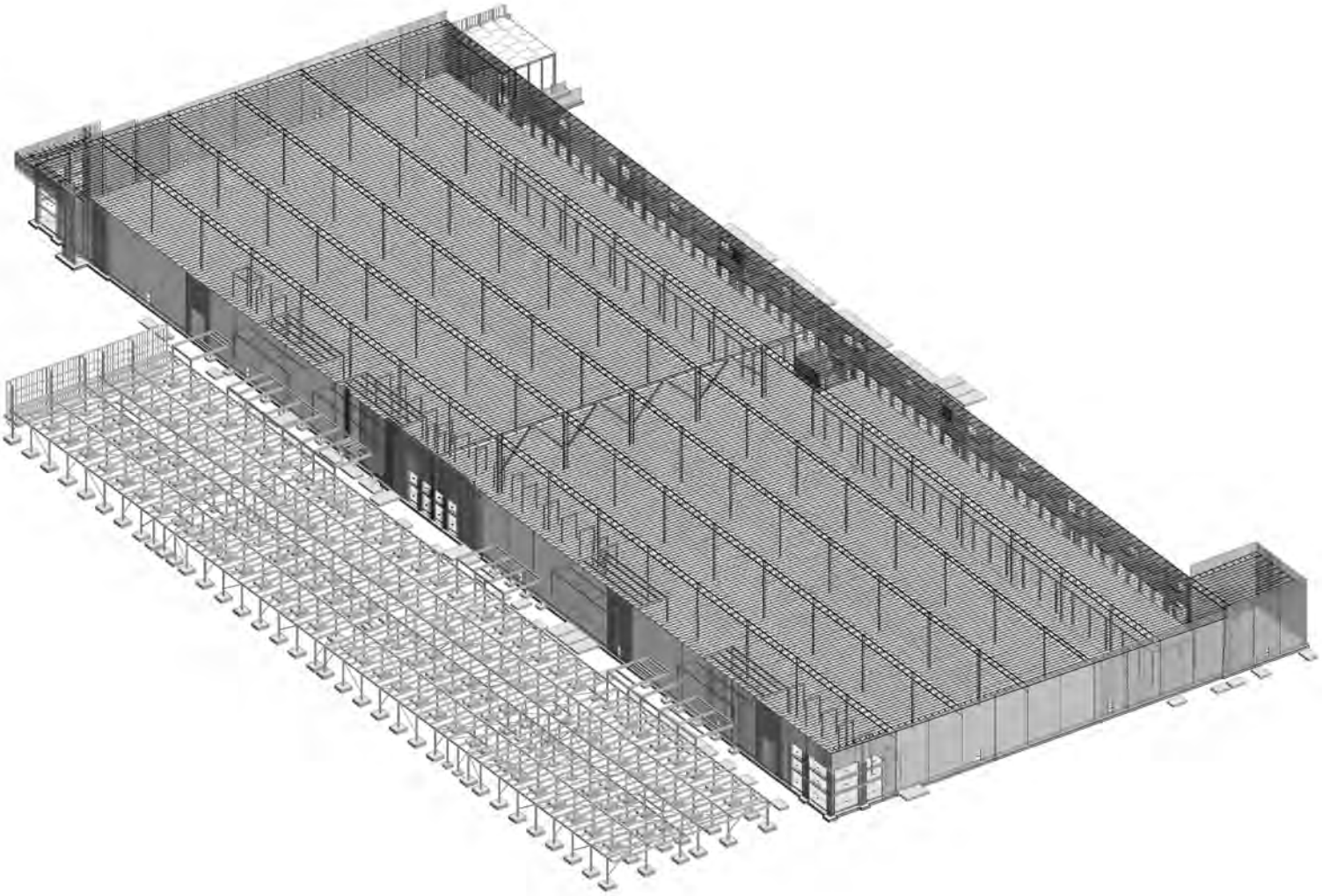
Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.39			
Ctrl Eq.	:	D13 at 40.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.39	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	17.59	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	17.59	Cl.H1.1
Available axial strength (P_c)	[Kip]	96.22	Cl.H1.1

Chapter 6 - Water Tank Equipment Supports

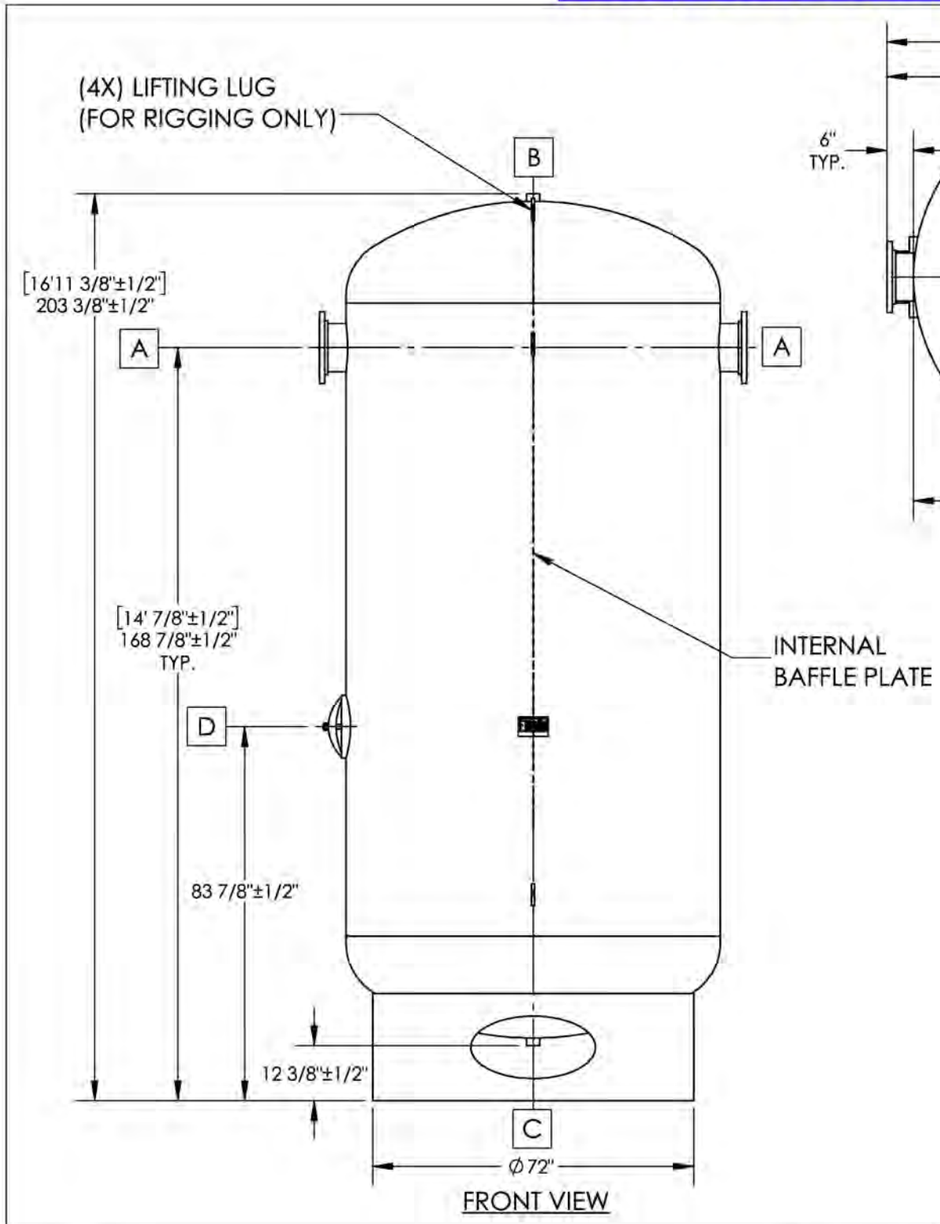


WATER TANK CUT SHEET PER MANUFACTURER

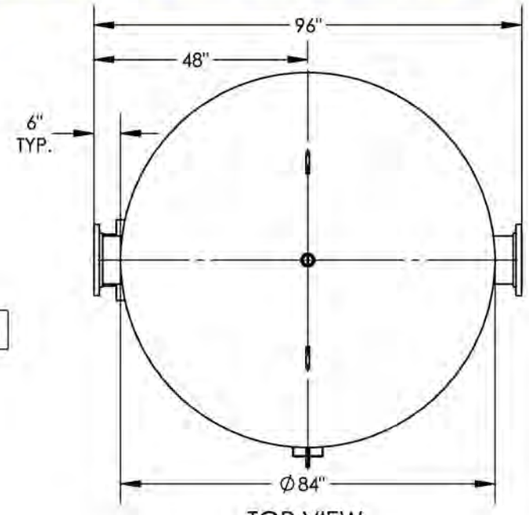
REV	DATE	ECO	DR BY	APP ENG	APP MFG	APP QA
A	9/29/2025		RLS	AEB	AEB	

TAG: Chilled Wtr Buffer - DATA HALL

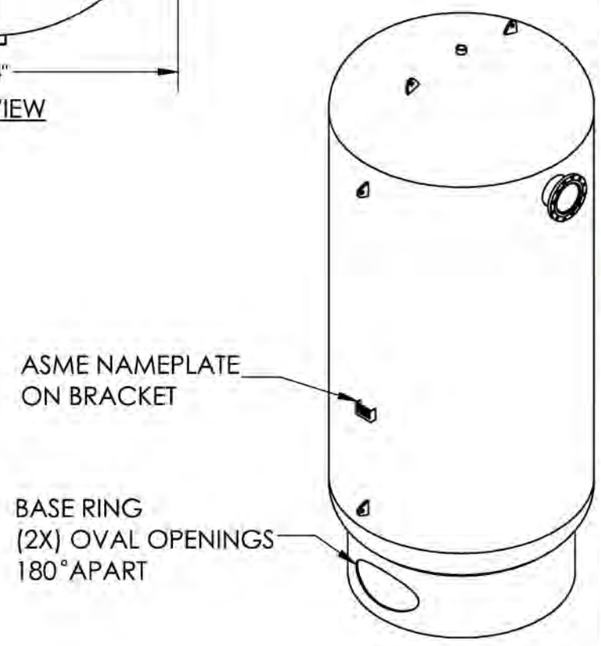
QTY: [6] - Auburndale FLA



FRONT VIEW



TOP VIEW



ISOMETRIC VIEW

DESIGN DATA		
CONSTRUCTION CODE	ASME SECTION VIII,DIVISION1,LATEST EDITION	
DESIGN PRESSURE	150 PSI	
TEST PRESSURE	195 PSI	
DESIGN TEMPERATURE	375 °F	
MDMT	-20 °F @ 150 PSI	
CORROSION ALLOWANCE	NONE	
SEISMIC CODE	NONE	
EMPTY WEIGHT (TOTAL)	FLOODED WEIGHT	CAPACITY
10,059 LBS	43,419 LBS	4,000 GAL.
EXTERNAL FINISH	ONE COAT RED OXIDE PRIMER	
INSULATION	NONE	
MATERIALS OF CONSTRUCTION		
HEADS	CARBON STEEL	
SHELL	CARBON STEEL	
COUPLINGS/FITTINGS	CARBON STEEL	
NOZZLES	CARBON STEEL	
BAFFLE	CARBON STEEL	
ATTACHMENTS	CARBON STEEL / 304SS	
BASE RING	CARBON STEEL	
NOZZLE SCHEDULE		
MARK/FUNCTION	SIZE	RATING TYPE
A (SYSTEM CONNECTION)	10"	CL150 ANSI RFSO
B (VENT)	2"	CL3000 NPT
C (DRAIN)	2"	CL3000 NPT
D (MANWAY)	12" X 16"	300 PSI

NOTES:

1. THIS IS NOT A CERTIFIED 'AS-BUILT' DRAWING, DO NOT USE DIMENSIONS FOR CONSTRUCTION
2. NOZZLES ARE NOT DESIGNED WITH EXTERNAL PIPE LOADS CONSIDERED.
3. FLANGE BOLT PATTERNS MUST STRADDLE PRINCIPLE CENTERLINES.
4. ENTIRE PERIMETER OF BASE RING TO BE IN CONTACT WITH SUITABLE SUPPORT PAD/STRUCTURE.

UNLESS OTHERWISE SPECIFIED:

FRACTIONS±1/4[.397]
.XX ± .02[.508]
.XXX ± .005[.127]
.XXXX ± .0005[.0127]
ANGLES ± 1°

FEATURE CONTROL SYMBOLS PER ASME Y14.5 2018

FINISHES 125
R.005[.127]TO.015[.38] BREAK SHARP EDGES
INCHES [MM]

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CUSTOMER: TERRONI, B.J.
REF. PART # BTH4000F10-150NN(SPECIAL)
SPA# 0925-1052 **PO # ---**
JOB# ---

DR BY	APP ENG	APP MFG	APP QA
RLS	AEB	AEB	

DATE	SCALE	SHEET
9/29/2025	1:48	1 OF 1

ECO/Submittal: 5447
VOL: SEE DESIGN DATA

Taco Inc. FALL RIVER, MA.

BUFFER TANK VERTICAL HIGH 84" DIA. 150PSI

SIZE		REV
B	BTH4000-M003	A

6.2

Released

Chavez-Grieves
consulting engineers, inc.
4700 LINCOLN RD NE STE 102 • ALBUQUERQUE, NM 87109
PHONE (505) 344-4080 • FAX (505) 343-8759

SHEET NO. _____ OF _____
JOB PROJECT
SUBJECT SUBJECT
CLIENT CLIENT
JOB NO. PROJECT NUM
BY FA DATE DATE

WATER TANK DESIGN LOADS

WIND LOAD FOR MULTIPLE TANKS (ASCE 7-22: CHAPTER 29)

$K_z = 0.85$ (Table 26.10-1)
$K_{zt} = 1$ (Chapter 26.8)
$K_e = 1$ (Table 26.9-1)
$V = 140$ mph
$q_z = 0.00256 \times K_z \times K_{zt} \times K_e \times V^2$ (Eq. 26.10-1)
$q_z = 42.65$psf

$K_d = 1$ (Table 26.6-1, Round Tank)
$G = 0.85$
$C_f = 1.1$ (Figure 29.4-6, Grouped Tanks)
$h = 17$ft
$D = 43.5$ft
$A_f = 17\text{ft} \times 43.5\text{ft} = 739.5\text{ft}^2$
$F = q_z \times K_d \times G \times C_f \times A_f$ (Eq. 29.4-1)
$F_p = 29.5$kips

WIND LOAD FOR SINGLE TANKS (ASCE7-22: CHAPTER 29)

$K_z = 0.85$ (Table 26.10-1)
$K_{zt} = 1$ (Chapter 26.8)
$K_e = 1$ (Table 26.9-1)
$V = 140$ mph
$q_z = 0.00256 \times K_z \times K_{zt} \times K_e \times V^2$ (Eq. 26.10-1)
$q_z = 42.65$psf

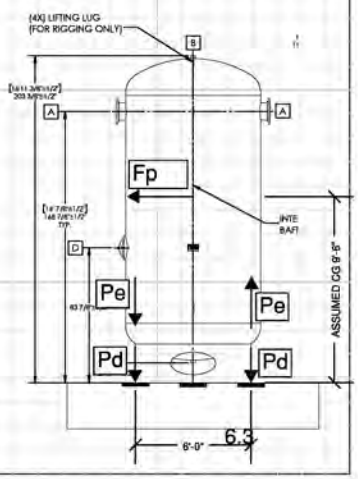
$K_d = 1$ (Table 26.6-1, Round Tank)
$G = 0.85$
$C_f = 0.5$ (Figure 29.4-1)
$h = 17$ft
$D = 7$ft
$A_f = 17\text{ft} \times 7\text{ft} = 119\text{ft}^2$
$F = q_z \times K_d \times G \times C_f \times A_f$ (Eq. 29.4-1)
$F_p = 2.16$kips

SEISMIC LATERAL FORCE FOR FILLED TANK

WEIGHT = 43.5kips (Weight per tank cut sheets)
EQ = 4.35kips

SEISMIC LATERAL FORCE FOR EMPTY TANK

WEIGHT = 10.1kips (Weight per tank cut sheets)
EQ = 1.01kips



LOADS WHEN TANK IS FULL

Pe = (29.5kips/5) x 9.5ft / 6ft) = 9.34kips
Pu = Pd - Pe = (43.5k/2)-9.34k = 12.5kips (no uplift)

Pu = Pd + Pe = (43.5k/2)+9.34k = 31.25kips (no uplift)

LOADS WHEN TANK IS EMPTY

Pu = Pd - Pe = (10.1k/2)-9.34 = -4.5kip (uplift)

Pu = Pd + Pe = (10.1k/2)+9.34k = 14.5kips (no uplift)

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Water Tank pad.ec6

LIC# : KW-06018138, Build:20.26.04.17 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Empty water tanks with moment on x dir

Code References

Governing Code : IBC 2024
Referenced Design Standard(s) : ACI 318-19
Load Combinations Used : ASCE 7-22 / IBC 2024

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	3.0 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	2.0 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

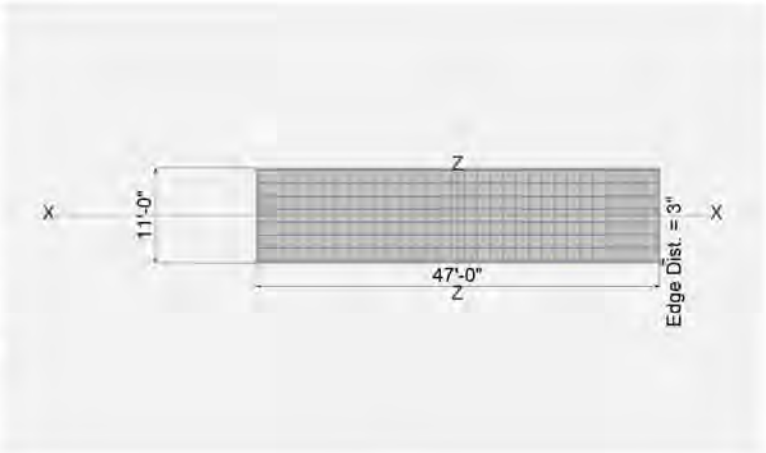
Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	47.0 ft
Length parallel to Z-Z Axis	=	11.0 ft
Footing Thickness	=	24.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	516.0 in
pz : parallel to Z-Z Axis	=	84.0 in
Height	=	in



Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation Bars along Z-Z Axis

# Bars required within zone	37.9 %
# Bars required on each side of zone	62.1 %

Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

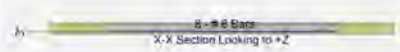
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 8

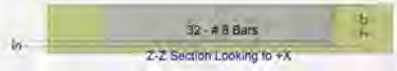
Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	32.0
Reinforcing Bar Size	=	# 8

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in





Top Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=	8.0
Reinforcing Bar Size	=	8

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	32.0
Reinforcing Bar Size	=	8

Rebar Centerline to Edge of Concrete...

at Top of footing	=	3.0 in

Weight = 10.1 kips x 5 = 50.5 kips

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Water Tank pad.ec6

LIC# : KW-06018138, Build:20.26.04.17 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Empty water tanks with moment on x dir

Applied Loads

		D	Lr	L	S	W	E	H
P : Vertical Load	=	50.500	0.000	0.000	0.000	0.000	0.000	0.000 k
OB : Overburden	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 ksf
M-xx	=	0.000	0.000	0.000	0.000	281.000	48.000	0.000 k-ft
M-zz	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
V-x	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k
V-z	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k

Fp = 29.5kips
Mx_w = 29.5kips x 9.5ft = 280.25kip-ft

EQ = 1.01kips x 5 = 5.05kips
Mx_{eq} = 48kip-ft

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.1879	Soil Bearing	0.5638 ksf	3.0 ksf	+D+0.60W about X-X axis
PASS	3.923	Overturning - X-X	168.60 k-ft	661.42 k-ft	+0.60D+0.60W
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	999.0	Sliding - X-X	0.0 k	65.629 k	D Only
PASS	999.0	Sliding - Z-Z	0.0 k	83.629 k	D Only
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.005158	Z Flexure (+X) Bot Tens	0.2725 k-ft/ft	52.838 k-ft/ft	+1.40D
PASS	0.005158	Z Flexure (-X) Bot Tens	0.2725 k-ft/ft	52.838 k-ft/ft	+1.40D
PASS	0.902	Min Steel X Flexure Bottom	0.518 in2/ft	0.575 in2/ft	n/a
PASS	0.01524	X Flexure (+Z) Bot Tens	0.7553 k-ft/ft	49.553 k-ft/ft	+1.20D+W
PASS	0.005519	X Flexure (-Z) Bot Tens	0.2735 k-ft/ft	49.553 k-ft/ft	+1.40D
PASS	0.964	Min Steel Z Flexure Bottom	0.518 in2/ft	0.538 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.006965	X Flexure (-Z) Top Tens	0.3451 k-ft/ft	49.553 k-ft/ft	+0.90D+W
PASS	0.964	Min Steel Z Flexure Top	0.518 in2/ft	0.538 in2/ft	n/a
PASS	0.005897	1-way Shear (+X)	0.2551 psi	43.253 psi	+1.40D
PASS	0.005897	1-way Shear (-X)	0.2551 psi	43.253 psi	+1.40D
PASS	0.008413	1-way Shear (+Z)	0.3560 psi	42.313 psi	+1.20D+W
PASS	0.004181	1-way Shear (-Z)	0.1769 psi	42.313 psi	+0.90D+W
PASS	0.005586	2-way Punching	0.6083 psi	108.908 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	3.0	n/a	0.0	0.3877	0.3877	n/a	n/a	0.129
X-X, +D+0.60W	3.0	n/a	10.094	0.2116	0.5638	n/a	n/a	0.188
X-X, +D+0.450W	3.0	n/a	7.571	0.2556	0.5198	n/a	n/a	0.173
X-X, +0.60D+0.60W	3.0	n/a	16.824	0.05651	0.4087	n/a	n/a	0.136
X-X, +D+0.70E	3.0	n/a	2.012	0.3526	0.4228	n/a	n/a	0.141
X-X, +D+0.5250E	3.0	n/a	1.509	0.3614	0.4140	n/a	n/a	0.138
X-X, +0.60D+0.70E	3.0	n/a	3.353	0.1975	0.2677	n/a	n/a	0.089
Z-Z, D Only	3.0	0.0	n/a	n/a	n/a	0.3877	0.3877	0.129
Z-Z, +D+0.60W	3.0	0.0	n/a	n/a	n/a	0.3877	0.3877	0.129
Z-Z, +D+0.450W	3.0	0.0	n/a	n/a	n/a	0.3877	0.3877	0.129
Z-Z, +0.60D+0.60W	3.0	0.0	n/a	n/a	n/a	0.2326	0.2326	0.078
Z-Z, +D+0.70E	3.0	0.0	n/a	n/a	n/a	0.3877	0.3877	0.129
Z-Z, +D+0.5250E	3.0	0.0	n/a	n/a	n/a	0.3877	0.3877	0.129
Z-Z, +0.60D+0.70E	3.0	0.0	n/a	n/a	n/a	0.2326	0.2326	0.078

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft) Overturning	Resisting	Ratio	Status	Moment Sum About Positive Edge (k-ft) Overturning	Resisting	Ratio	Status
X-X, D Only	None	1,102.37	Infinity	OK	None	1,102.37	Infinity	OK
X-X, +D+0.60W	None	1,270.97	Infinity	OK	168.60	1,102.37	6.538	OK
X-X, +D+0.450W	None	1,228.82	Infinity	OK	126.45	1,102.37	8.718	OK
X-X, +0.60D+0.60W	None	830.02	Infinity	OK	168.60	661.42	3.923	OK

6.5

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Water Tank pad.ec6

LIC# : KW-06018138, Build:20.26.04.17 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Empty water tanks with moment on x dir

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft)				Moment Sum About Positive Edge (k-ft)			
	Overturning	Resisting	Ratio	Status	Overturning	Resisting	Ratio	Status
X-X, +D+0.70E	None	1,135.97	Infinity	OK	33.60	1,102.37	32.808	OK
X-X, +D+0.5250E	None	1,127.57	Infinity	OK	25.20	1,102.37	43.745	OK
X-X, +0.60D+0.70E	None	695.02	Infinity	OK	33.60	661.42	19.685	OK
Z-Z, D Only	None	4,710.11	Infinity	OK	None	4,710.11	Infinity	OK
Z-Z, +D+0.60W	None	4,710.11	Infinity	OK	None	4,710.11	Infinity	OK
Z-Z, +D+0.450W	None	4,710.11	Infinity	OK	None	4,710.11	Infinity	OK
Z-Z, +0.60D+0.60W	None	2,826.06	Infinity	OK	None	2,826.06	Infinity	OK
Z-Z, +D+0.70E	None	4,710.11	Infinity	OK	None	4,710.11	Infinity	OK
Z-Z, +D+0.5250E	None	4,710.11	Infinity	OK	None	4,710.11	Infinity	OK
Z-Z, +0.60D+0.70E	None	2,826.06	Infinity	OK	None	2,826.06	Infinity	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.2735	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.40D	0.2735	-Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D	0.2344	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D	0.2344	-Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D+0.50W	0.4949	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D+W	0.7553	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +0.90D+W	0.6967	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D+E	0.3234	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D+E	0.1454	-Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +0.90D+E	0.2648	+Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +0.90D+E	0.08682	-Z	Bottom	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
Z-Z, +1.40D	0.2725	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.40D	0.2725	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D	0.2336	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D	0.2336	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+0.50W	0.2336	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+0.50W	0.2336	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+W	0.2336	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+W	0.2336	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +0.90D+W	0.1752	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +0.90D+W	0.1752	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+E	0.2336	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +1.20D+E	0.2336	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +0.90D+E	0.1752	-X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK
Z-Z, +0.90D+E	0.1752	+X	Bottom	0.5184	ACI 7.6.1.1	0.5745	52.838	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.20D+0.50W	0.02605	-Z	Top	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +1.20D+W	0.2865	-Z	Top	0.5184	ACI 7.6.1.1	0.5379	49.553	OK
X-X, +0.90D+W	0.3451	-Z	Top	0.5184	ACI 7.6.1.1	0.5379	49.553	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.26 psi	0.26 psi	0.26 psi	43.25 psi	0.01	OK
+1.20D	0.22 psi	0.22 psi	0.22 psi	43.25 psi	0.01	OK
+1.20D+0.50W	0.22 psi	0.22 psi	0.22 psi	43.25 psi	0.01	OK
+1.20D+W	0.22 psi	0.22 psi	0.22 psi	43.25 psi	0.01	OK
+0.90D+W	0.16 psi	0.16 psi	0.16 psi	43.25 psi	0.00	OK
+1.20D+E	0.22 psi	0.22 psi	0.22 psi	43.25 psi	0.01	OK
+0.90D+E	0.16 psi	0.16 psi	0.16 psi	43.25 psi	0.00	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.12 psi	0.12 psi	0.12 psi	42.31 psi	0.00	OK

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Water Tank pad.ec6

LIC# : KW-06018138, Build:20.26.04.17 CHAVEZ-GRIEVES CONSULTING (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Empty water tanks with moment on x dir

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.20D	0.10 psi	0.10 psi	0.10 psi	42.31 psi	0.00	OK
+1.20D+0.50W	0.02 psi	0.23 psi	0.23 psi	42.31 psi	0.01	OK
+1.20D+W	0.15 psi	0.36 psi	0.36 psi	42.31 psi	0.01	OK
+0.90D+W	0.18 psi	0.33 psi	0.33 psi	42.31 psi	0.01	OK
+1.20D+E	0.06 psi	0.15 psi	0.15 psi	42.31 psi	0.00	OK
+0.90D+E	0.03 psi	0.12 psi	0.12 psi	42.31 psi	0.00	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	0.61 psi	108.91 psi	0.01	OK
+1.20D	0.52 psi	108.91 psi	0.00	OK
+1.20D+0.50W	0.52 psi	108.91 psi	0.00	OK
+1.20D+W	0.52 psi	108.91 psi	0.00	OK
+0.90D+W	0.39 psi	108.91 psi	0.00	OK
+1.20D+E	0.52 psi	108.91 psi	0.00	OK
+0.90D+E	0.39 psi	108.91 psi	0.00	OK

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

Specifier's comments:

1 Input data



Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (6 1/4)**
Item number:	418085 KH-EZ 3/4"x7"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 6.25 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 4.840 in., h_{nom} = 6.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate[R]:	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.250 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 24.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

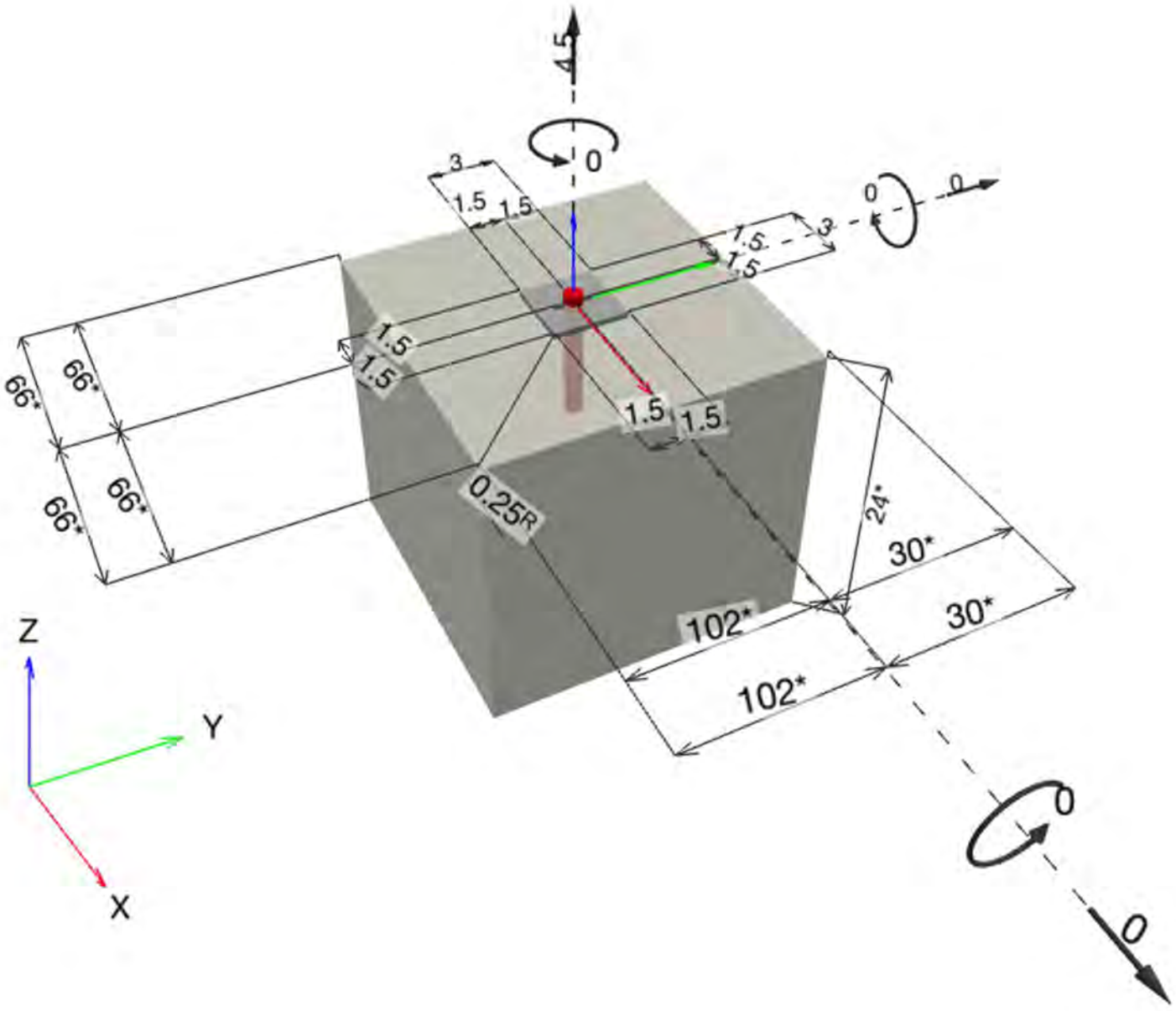
Application also possible with KWIK-X (Carbon Steel) 3/4 (4 1/2) hnom2 under the selected boundary conditions.
More information in section Alternative fastening data of this report.

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

Geometry [in.] & Loading [kip, ft.kip]



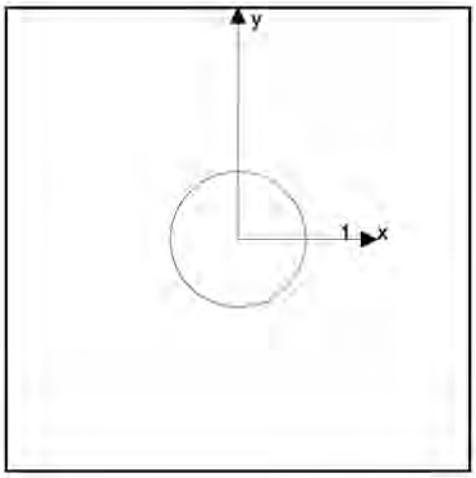
Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4.500; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	70

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	4.500	0.000	0.000	0.000

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(0.000/0.000): 4.500 [kip]
Resulting compression force in (x/y)=(-/-): 0.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	4.500	20.808	22	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	4.500	6.445	70	OK

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3027
$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

N_{sa} [kip]
32.013

Results

N_{sa} [kip]	ϕ_{steel}	$\phi\ N_{sa}$ [kip]	N_{ua} [kip]
32.013	0.650	20.808	4.500

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.840	30.000	1.000	7.280	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
210.83	210.83	1.000	1.000	9.915

Results

N_{cb} [kip]	$\phi_{concrete}$	$\phi\ N_{cb}$ [kip]	N_{ua} [kip]
9.915	0.650	6.445	4.500

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 20, 2026	Date:	5/21/2026
Fastening point:			

6 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (6 1/4)

Profile: no profile

Item number: 418085 KH-EZ 3/4"x7"

Hole diameter in the fixture: d_f = 0.875 in.

Maximum installation torque: 0.09501 ft.kip

Plate thickness (input): 0.250 in.

Hole diameter in the base material: 0.750 in.

Recommended plate thickness: not calculated

Hole depth in the base material: 6.625 in.

Drilling method: Hammer drilled

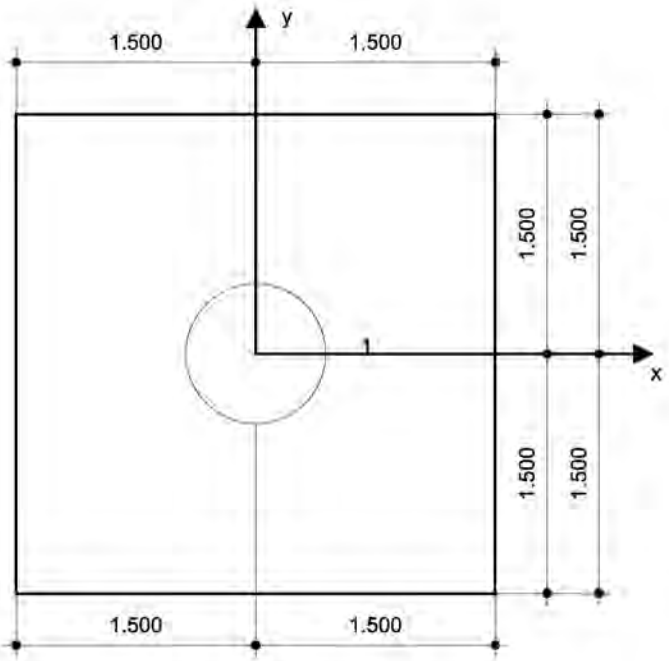
Minimum thickness of the base material: 8.125 in.

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 6.25 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench • Hilti SIW 9-A22 Impact Wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	66.000	66.000	102.000	30.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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7 Alternative fastening

7.1 Alternative fastening data



Anchor type and diameter:	**KWIK-X (Carbon Steel) 3/4 (4 1/2) hnom2**
Item number:	418084 KH-EZ 3/4"x5 1/2" (element) / 2346813 KHC 3/4" LARGE (capsule)
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 6.25 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 4.840 in. ($h_{ef,limit}$ = - in.), h_{nom} = 4.840 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-5065
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Chem
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate^R:	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.250 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Max. Utilization with KWIK-X (Carbon Steel) 3/4 (4 1/2) hnom2: 70 %
Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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7.2 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 0.250 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: KWIK-X (Carbon Steel) 3/4 (4 1/2) hnom2

Item number: 418084 KH-EZ 3/4"x5 1/2" (element) / 2346813 KHC 3/4" LARGE (capsule)

Maximum installation torque: -

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.840 in.

Minimum thickness of the base material: 7.340 in.

Hilti ∅ 3/4 in KWIK-X (Carbon Steel) with KH-EZ with KHC 3/4" LARGE with 4.84 in nominal embedment depth per ICC-ES ESR-5065, Hammer drill bit installation per MPII

7.2.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• No accessory required	• SIW 6-A22 Impact Screw Driver

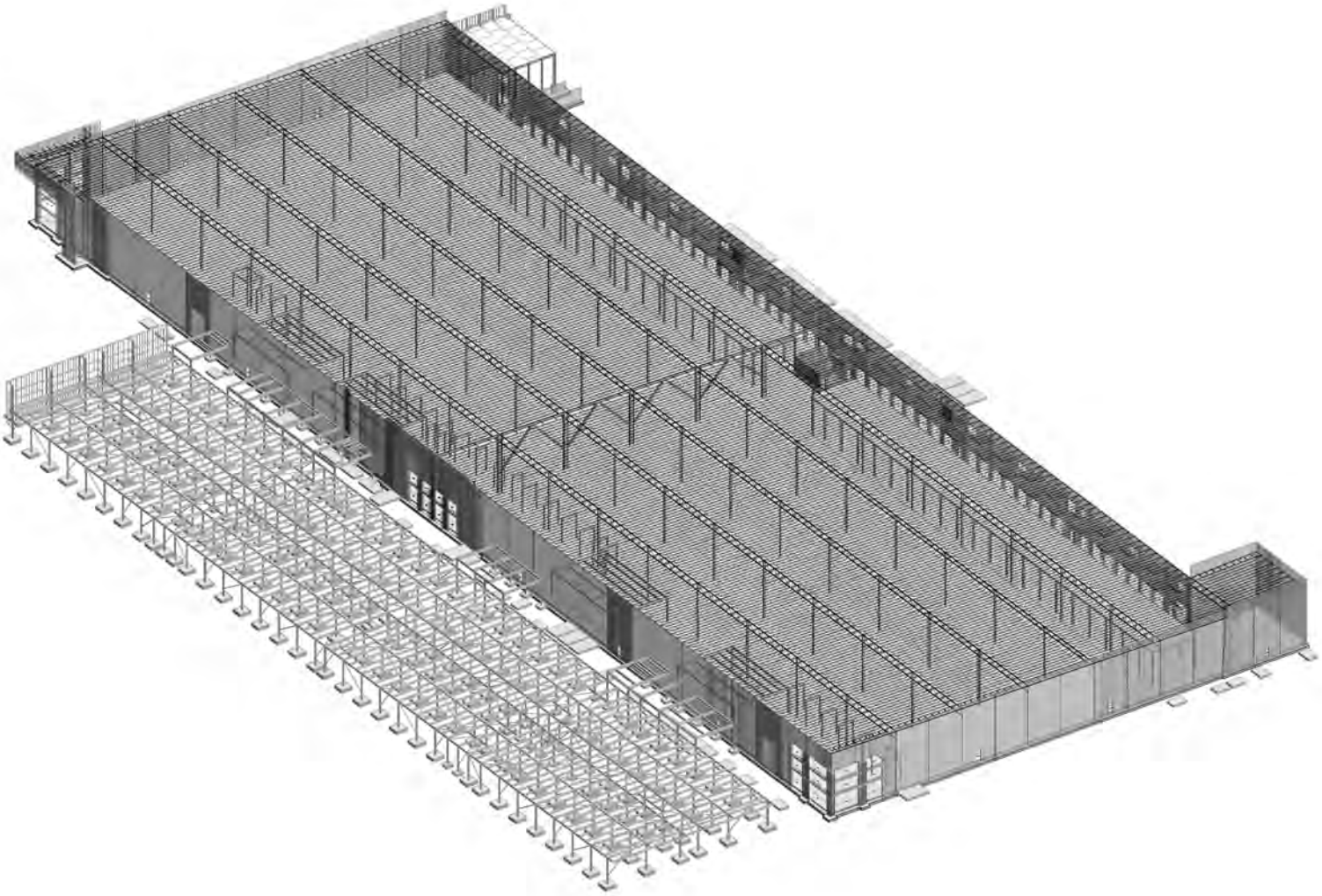
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8 Remarks; Your Cooperation Duties

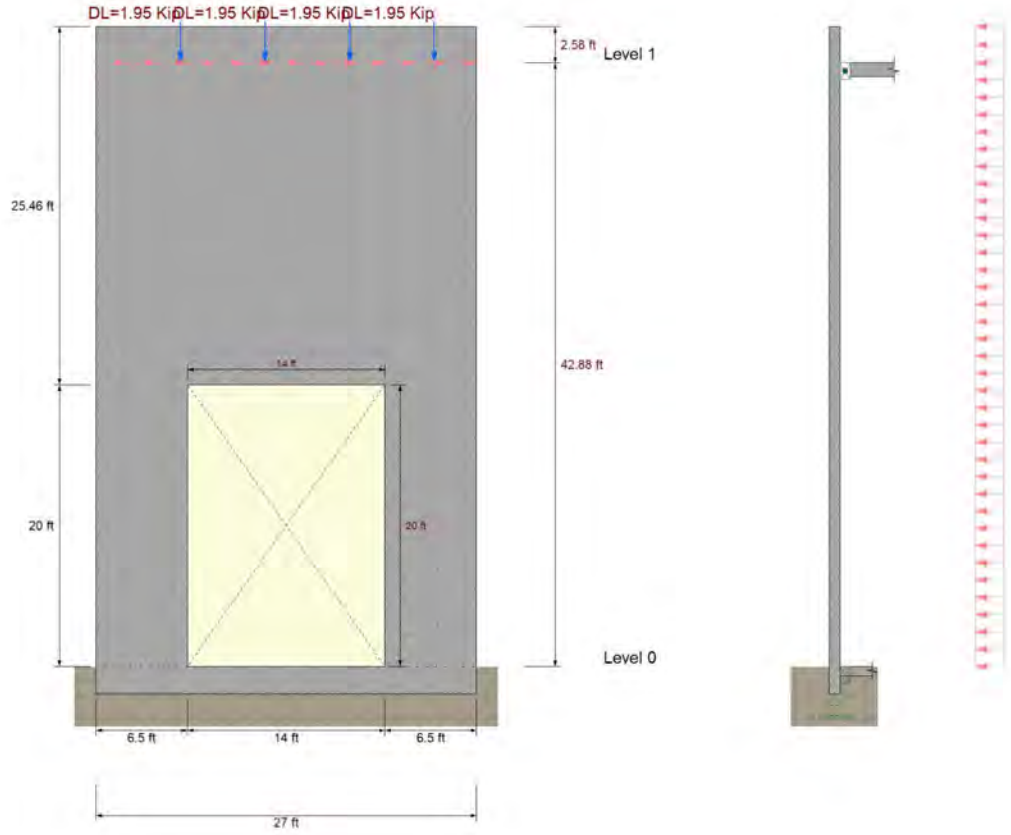
- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
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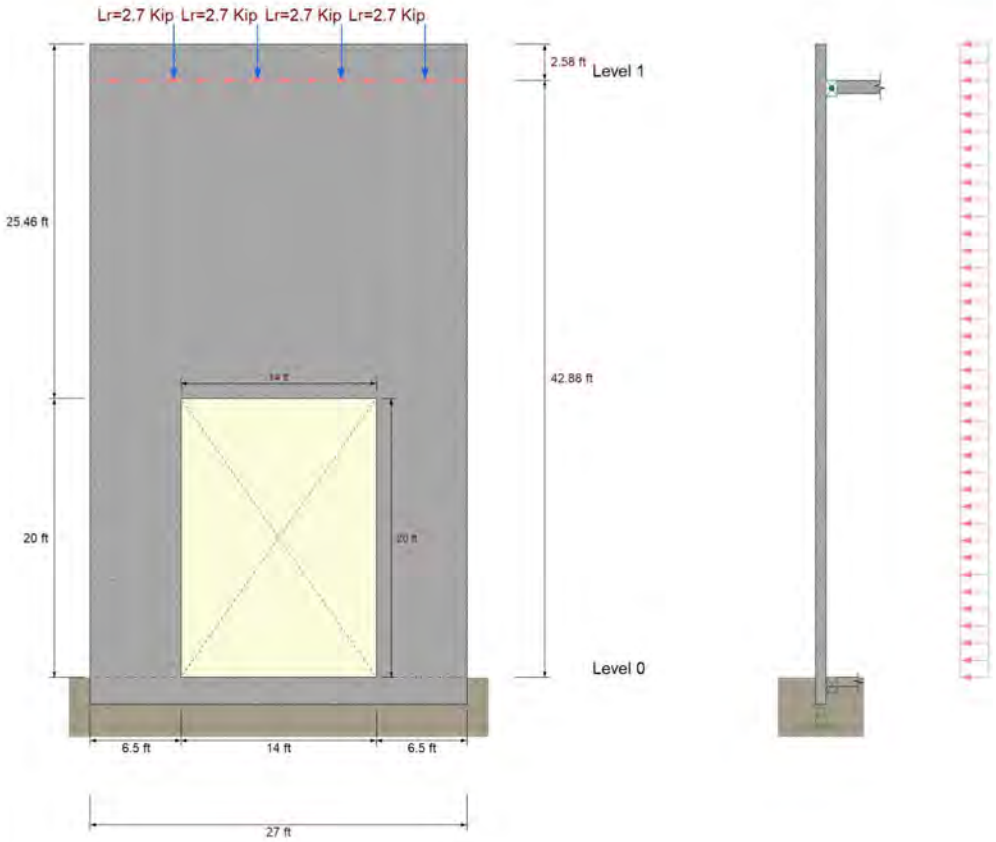
Chapter 7 - New Openings in Existing Tilt Up Concrete Walls



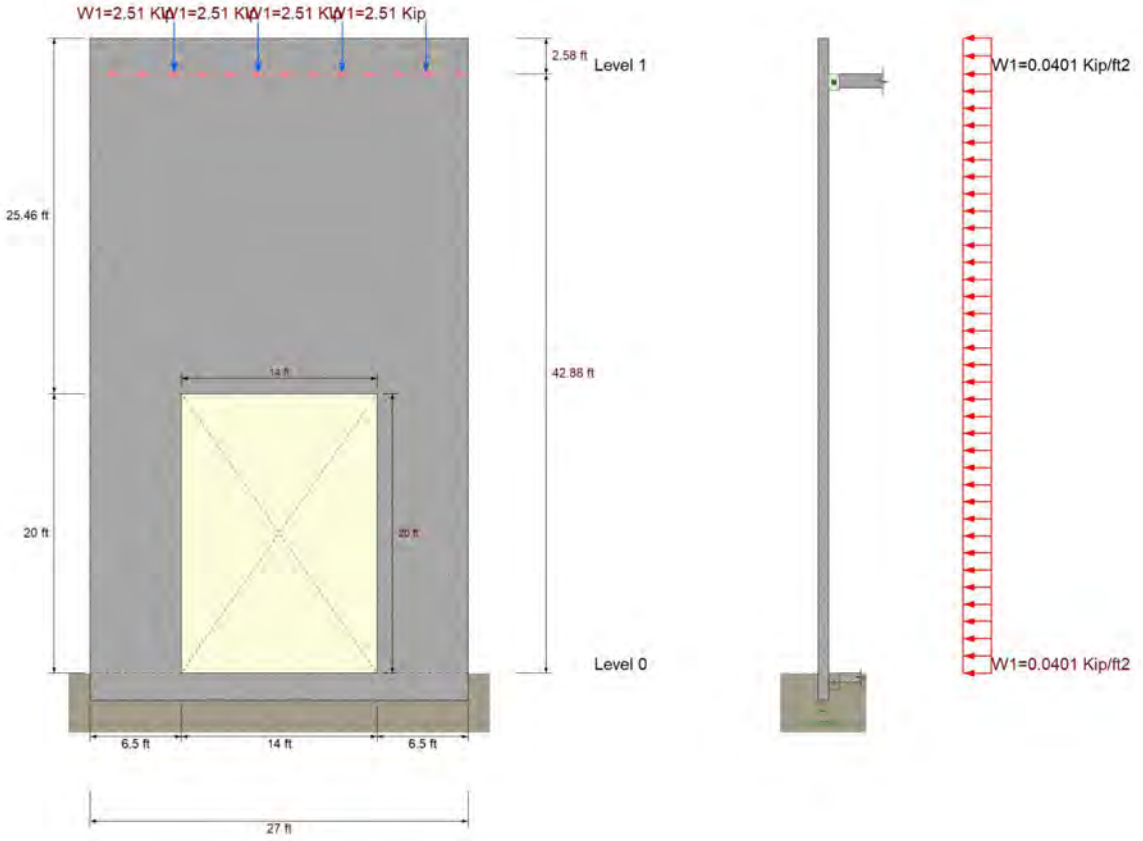
Existing Roof Dead Loads to New Opening in Existing Tilt Up Concrete Wall



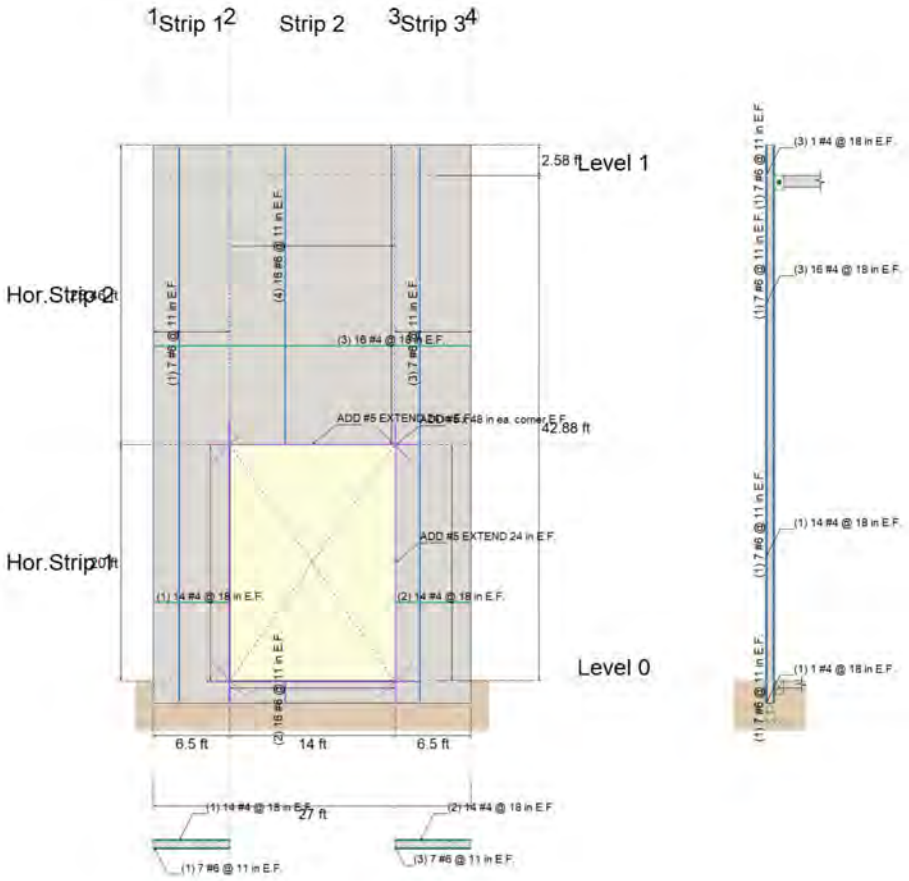
Existing Roof Live Loads to New Opening in Existing Tilt Up Concrete Wall



Wind Component and Cladding Loads to New Opening in Existing Tilt Up Concrete Wall



Reinforcement in Existing Tilt Up Concrete Wall



RAM Elements **Salas O'Brien**

Current Date: 5/29/2026 1:11 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\04.2 Existing Wall\New Overhead Door Opening Along Grid A Near Grid 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	47.38 [ft]
Total length	:	27.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	42.88	9.25

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.50	0.00	14.00	20.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
Lr	No	LLR	Roof Live Load
W1	No	WIND	Wind Load C&C
W2	No	WIND	Wind Load MWFRS
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5Lr

D3	Yes	1.2DL+1.6Lr
D4	Yes	1.2DL+0.5W1
D5	Yes	1.2DL+0.5W2
D6	Yes	1.2DL+1.6Lr+0.5W1
D7	Yes	1.2DL+1.6Lr+0.5W2
D8	Yes	1.2DL+W1
D9	Yes	1.2DL+W2
D10	Yes	1.2DL+0.5Lr+W1
D11	Yes	1.2DL+0.5Lr+W2
D12	Yes	0.9DL+W1
D13	Yes	0.9DL+W2
D14	Yes	DL
D15	Yes	DL+Lr
D16	Yes	DL+0.75Lr
D17	Yes	DL+0.6W1
D18	Yes	DL+0.6W2
D19	Yes	DL+0.75Lr+0.45W1
D20	Yes	DL+0.75Lr+0.45W2
D21	Yes	0.6DL+0.6W1
D22	Yes	0.6DL+0.6W2
S1	Yes	DL
S2	Yes	DL+Lr
S3	Yes	DL+0.75Lr
S4	Yes	DL+0.6W1
S5	Yes	DL+0.6W2
S6	Yes	DL+0.45W1+0.75Lr
S7	Yes	DL+0.45W2+0.75Lr
S8	Yes	0.6DL+0.6W1
S9	Yes	0.6DL+0.6W2

Loads

Consider Self Weight:

Load condition : DL

Concentrated loads:

Story	Condition	Direction	Magnitude [Kip]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	1.95	0.00	6.00
1	Lr	Vertical	2.70	0.00	6.00
1	W1	Vertical	2.51	0.00	6.00
1	DL	Vertical	1.95	0.00	12.00
1	Lr	Vertical	2.70	0.00	12.00
1	W1	Vertical	2.51	0.00	12.00
1	DL	Vertical	1.95	0.00	18.00
1	Lr	Vertical	2.70	0.00	18.00
1	W1	Vertical	2.51	0.00	18.00
1	DL	Vertical	1.95	0.00	24.00
1	Lr	Vertical	2.70	0.00	24.00
1	W1	Vertical	2.51	0.00	24.00

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	W2	Horizontal	0.30	0.00

Out-of-plane pressure loads:

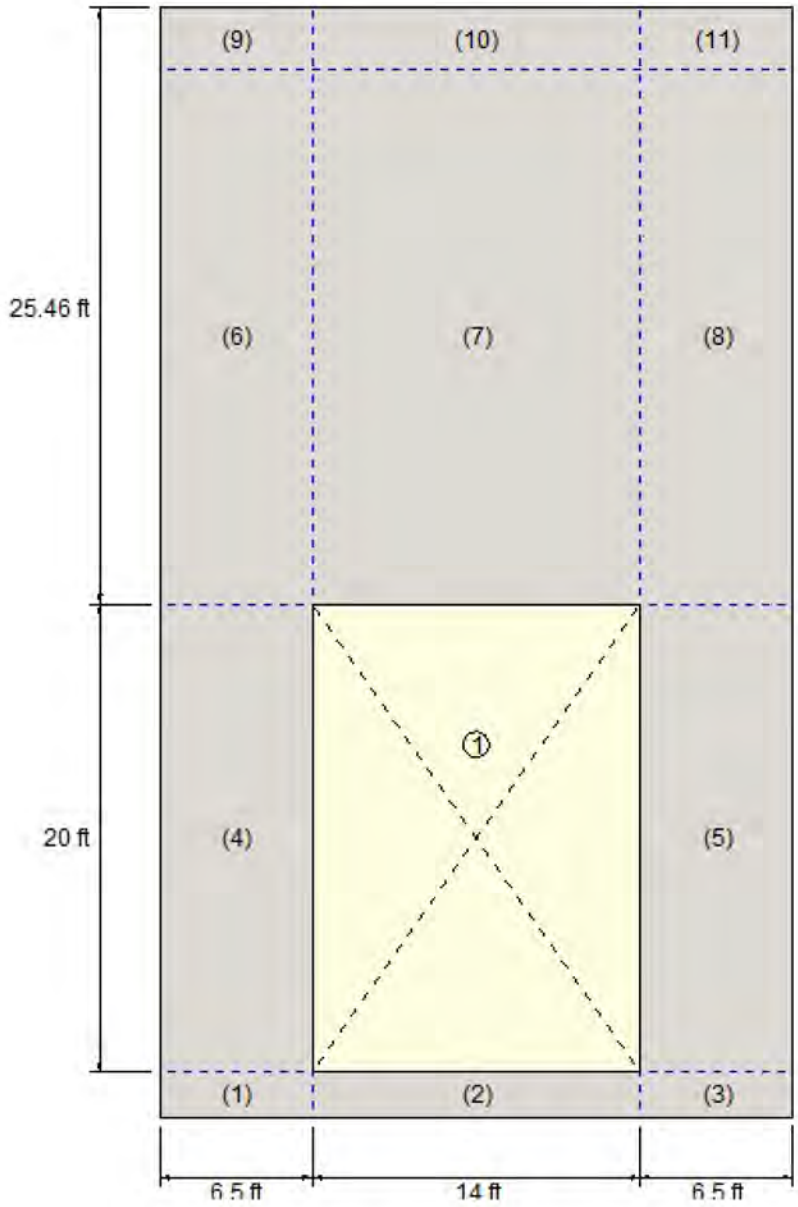
Story	Condition	Magnitude [Kip/ft2]
1	W1	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
W1	0.04

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 4)
- Excessive deflection, Section 11.8.1.1e (Segment 4)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	-1.92	6.50	1.92
2	6.50	-1.92	14.00	1.92
3	20.50	-1.92	6.50	1.92
4	0.00	0.00	6.50	20.00
5	20.50	0.00	6.50	20.00
6	0.00	20.00	6.50	22.88
7	6.50	20.00	14.00	22.88
8	20.50	20.00	6.50	22.88
9	0.00	42.88	6.50	2.58
10	6.50	42.88	14.00	2.58
11	20.50	42.88	6.50	2.58

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#6	11.00	28.46
2	16-#6	11.00	28.46
3	7-#6	11.00	28.46
4	7-#6	11.00	28.46
5	7-#6	11.00	28.46
6	7-#6	11.00	28.46
7	16-#6	11.00	28.46
8	7-#6	11.00	28.46
9	7-#6	11.00	28.46
10	16-#6	11.00	28.46
11	7-#6	11.00	28.46

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D1 (Max)	80.638	0.112	0.240	0.47
2	D1 (Max)	2.851	0.002	0.240	0.01
3	D11 (Max)	84.937	0.118	0.240	0.49
4	D6 (Max)	81.462	0.113	0.240	0.47
5	D6 (Max)	81.462	0.113	0.240	0.47
6	D6 (Max)	39.896	0.055	0.240	0.23
7	D6 (Max)	18.057	0.012	0.240	0.05
8	D6 (Max)	39.896	0.055	0.240	0.23
9	D4 (Max)	2.319	0.003	0.240	0.01
10	D4 (Max)	4.996	0.003	0.240	0.01
11	D4 (Max)	2.319	0.003	0.240	0.01

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D12 (Top)	3.65	0.91	1.07	7.63	0.90	93885.15
2	D1 (Top)	7.07	0.74	0.88	7.63	0.90	724867.40
3	D12 (Top)	3.65	0.91	1.07	7.63	0.90	93885.15
4	D10 (Max)	3.87	0.99	1.17	7.63	0.90	193.80
5	D10 (Max)	3.87	0.99	1.17	7.63	0.90	193.80
6	D10 (Max)	3.46	0.84	0.99	7.63	0.90	218.99
7	D12 (Max)	7.16	0.76	0.89	7.63	0.90	1448.63
8	D10 (Max)	3.46	0.84	0.99	7.63	0.90	218.99
9	D12 (Bottom)	3.10	0.70	0.83	7.63	0.90	185268.90
10	D12 (Bottom)	7.08	0.75	0.88	7.63	0.90	399040.70
11	D12 (Bottom)	3.10	0.70	0.83	7.63	0.90	185268.90

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D12 (Top)	5144.45	1294.34	1435.13
2	D1 (Top)	11080.34	2627.84	11080.34
3	D12 (Top)	5144.45	1294.34	1435.13
4	D10 (Max)	5144.45	1339.61	1482.34
5	D10 (Max)	5144.45	1339.61	1482.34
6	D10 (Max)	5144.45	1251.42	1675.00
7	D12 (Max)	11080.34	2648.31	11080.34
8	D10 (Max)	5144.45	1251.42	1675.00
9	D12 (Bottom)	5144.45	1166.10	5144.45
10	D12 (Bottom)	11080.34	2629.98	11080.34
11	D12 (Bottom)	5144.45	1166.10	5144.45

Deflection

Segment	Condition	Δu [in]
1	D21 (Max)	0.007
2		0.000
3	D21 (Max)	0.007
4	D21 (Max)	4.467
5	D21 (Max)	4.467
6	D21 (Max)	-0.935
7	D21 (Max)	-0.239
8	D21 (Max)	-0.935
9	D21 (Max)	0.000
10	D21 (Max)	0.000
11	D21 (Max)	0.000

This requires the wall to be retrofitted with HSS strong backs

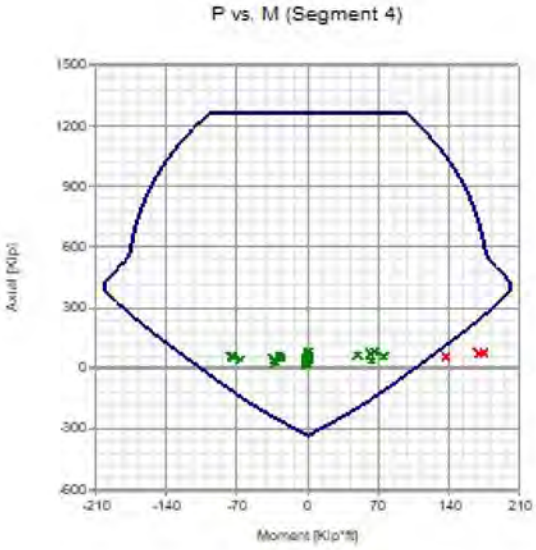
Combined axial flexure

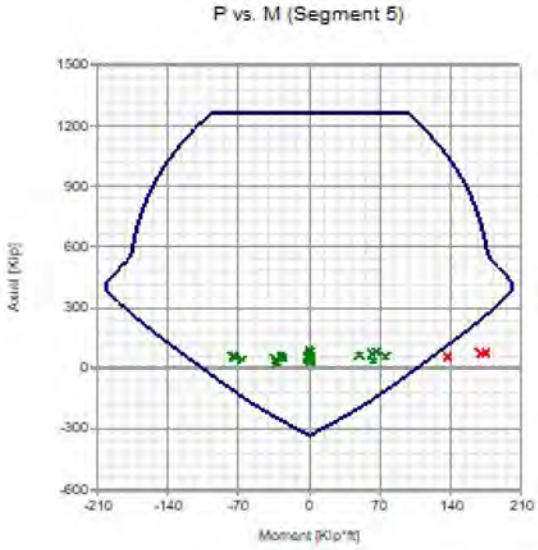
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D12 (Top)	56.77	80.96	81.03	121.31	0.67
2	D1 (Top)	2.85	0.00	0.00	240.52	0.00
3	D12 (Top)	56.77	80.96	81.03	121.31	0.67
4	D10 (Max)	78.03	81.24	175.39	126.98	1.38
5	D10 (Max)	78.03	81.24	175.39	126.98	1.38
6	D10 (Max)	37.53	-50.71	-65.72	116.09	0.57
7	D12 (Max)	11.46	-35.78	-36.17	242.85	0.15
8	D10 (Max)	37.53	-50.71	-65.72	116.09	0.57
9	D12 (Bottom)	1.74	0.86	0.86	106.38	0.01
10	D12 (Bottom)	3.75	1.86	1.86	240.76	0.01
11	D12 (Bottom)	1.74	0.86	0.86	106.38	0.01

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D22 (Top)	26.66	0.00	43.97	113.14	0.39
2	D22 (Max)	1.22	0.00	94.70	240.08	0.39
3	D21 (Top)	37.52	48.58	43.97	116.08	0.38
4	D22 (Top)	19.02	0.00	43.97	111.07	0.40
5	D21 (Top)	29.16	-29.77	43.97	113.82	0.39
6	D22 (Max)	4.85	0.00	43.97	107.22	0.41
7	D1 (Max)	-12.89	0.00	94.70	236.26	0.40
8	D21 (Top)	9.60	0.50	43.97	108.51	0.41
9	D21 (Top)	0.00	0.00	43.97	105.91	0.42
10	D21 (Top)	0.01	0.00	94.70	239.75	0.39
11	D21 (Top)	0.00	0.00	43.97	105.91	0.42

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Bottom)	82.63	1264.72	0.07
2	D1 (Bottom)	7.15	2722.58	0.00
3	D11 (Bottom)	86.65	1264.72	0.07
4	D1 (Bottom)	82.04	1264.72	0.06
5	D11 (Bottom)	84.92	1264.72	0.07
6	D6 (Bottom)	43.32	1264.72	0.03
7	D6 (Bottom)	40.23	2722.58	0.01
8	D7 (Bottom)	43.83	1264.72	0.03
9	D1 (Bottom)	2.71	1264.72	0.00
10	D1 (Bottom)	5.83	2722.58	0.00
11	D1 (Bottom)	2.71	1264.72	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Top)	0.00	332.64	0.00
2	D1 (Top)	0.00	760.32	0.00
3	D1 (Top)	0.00	332.64	0.00
4	D1 (Top)	0.00	332.64	0.00
5	D1 (Top)	0.00	332.64	0.00
6	D1 (Top)	0.00	332.64	0.00
7	D1 (Top)	12.89	760.32	0.02
8	D1 (Top)	0.00	332.64	0.00
9	D1 (Top)	0.00	332.64	0.00
10	D1 (Top)	0.00	760.32	0.00
11	D1 (Top)	0.00	332.64	0.00

Shear

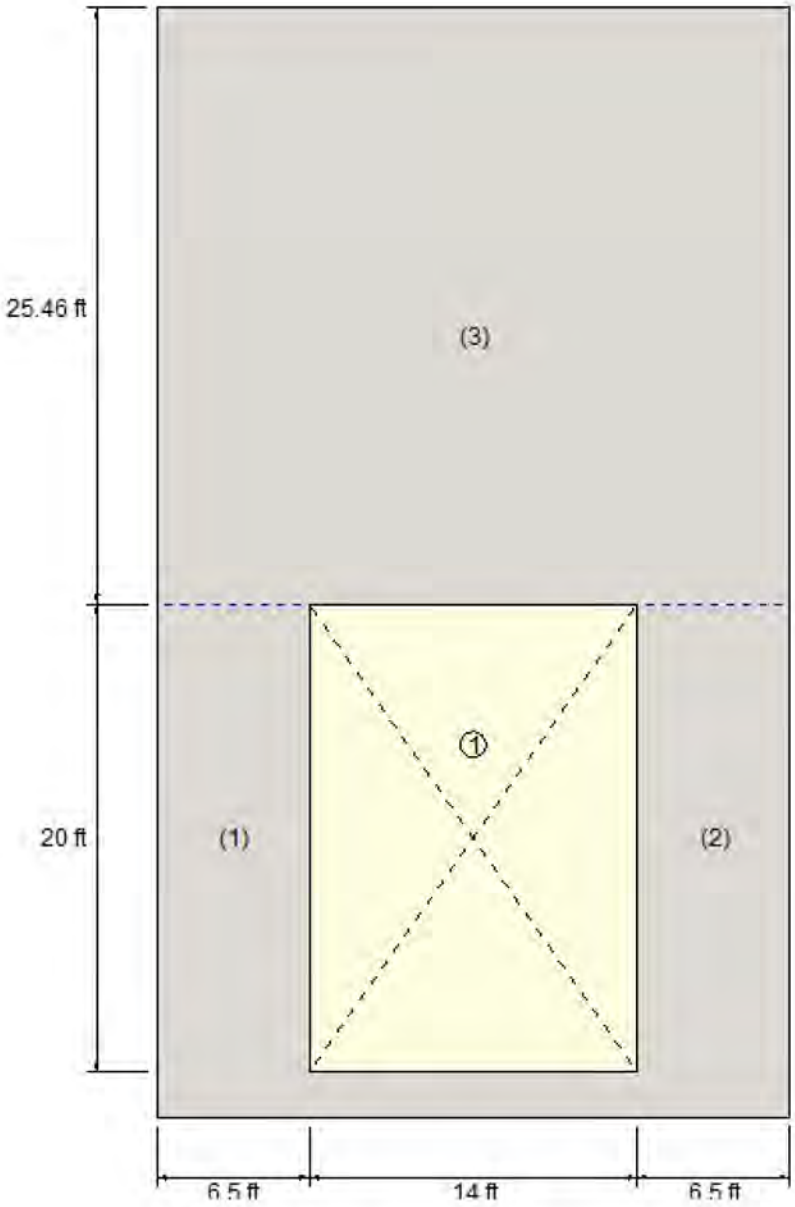
Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D12 (Max)	42.424	55.046	0.77
2	D1 (Top)	0.000	108.374	0.00
3	D12 (Max)	42.424	55.046	0.77
4	D12 (Max)	9.153	55.146	0.17
5	D12 (Max)	9.153	55.146	0.17
6	D12 (Top)	5.188	50.715	0.10
7	D8 (Top)	6.498	107.263	0.06
8	D12 (Top)	5.188	50.715	0.10
9	D12 (Max)	0.671	49.375	0.01
10	D12 (Max)	1.446	108.467	0.01
11	D12 (Max)	0.671	49.375	0.01

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	S8 (Max)	0.005	0.153	0.03
2	S1 (Top)	0.000	0.153	0.00
3	S8 (Max)	0.005	0.153	0.03
4	S4 (Max)	3.875	3.430	1.13
5	S4 (Max)	3.875	3.430	1.13
6	S4 (Max)	0.805	3.430	0.23
7	S4 (Max)	0.179	3.430	0.05
8	S4 (Max)	0.805	3.430	0.23
9	S8 (Max)	0.000	0.207	0.00
10	S8 (Max)	0.000	0.207	0.00
11	S8 (Max)	0.000	0.207	0.00

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	6.50	20.00	Shear wall
2	20.50	0.00	6.50	20.00	Shear wall
3	0.00	20.00	27.00	22.88	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	7-#6	11.00	28.46	14-#4	18.00	24.67
2	7-#6	11.00	28.46	14-#4	18.00	24.67
3	7-#6	11.00	28.46	16-#4	18.00	24.67
	16-#6	11.00	28.46	16-#4	18.00	24.67
	7-#6	11.00	28.46	16-#4	18.00	24.67

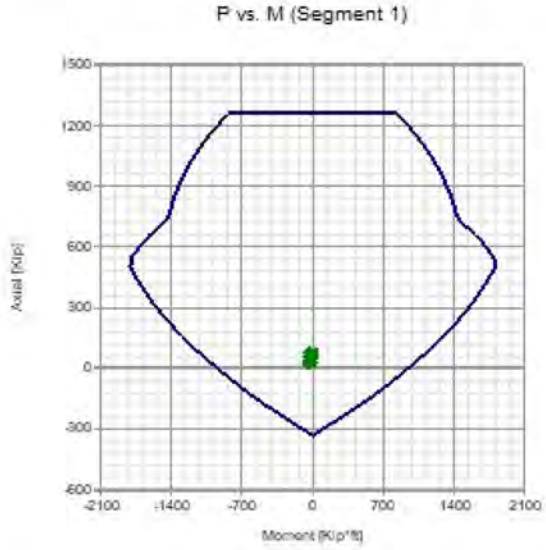
Intermediate results for axial-bending

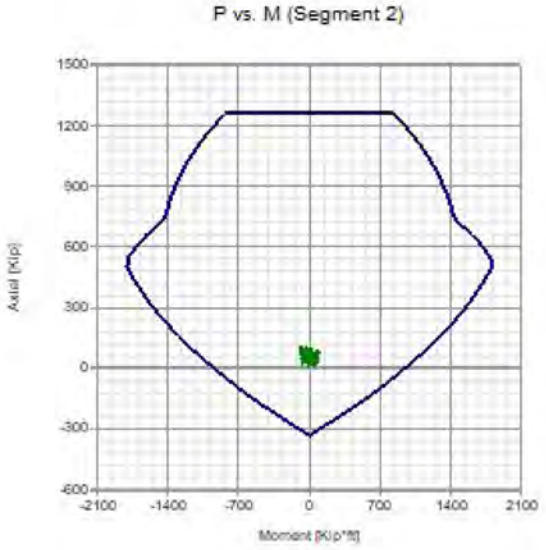
Segment	Condition	c [in]	d [in]
1	D13 (Bottom)	12.12	76.88
2	D11 (Top)	12.97	76.88
3	D13 (Bottom)	50.95	322.88

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D13 (Bottom)	40.83	-44.73	1040.17	0.04
2	D11 (Top)	66.92	63.93	1101.15	0.06
3	D13 (Bottom)	78.44	-185.22	17244.29	0.01

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Bottom)	82.04	1264.72	0.06
2	D11 (Bottom)	84.92	1264.72	0.07
3	D6 (Bottom)	126.88	5252.02	0.02

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Top)	0.00	332.64	0.00
2	D1 (Top)	0.00	332.64	0.00
3	D1 (Top)	0.00	1425.60	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	2.549	146.447	0.02
2	D11 (Top)	6.262	146.447	0.04
3	D9 (Top)	8.100	750.481	0.01

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
S1	Left corner	-1674.68	0.00	--
S9	Right corner	990.77	-208.37	4.75

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

Retrofit of Existing Tilt Up Concrete Wall with HSS Stiff Back



Member Sizes of Retrofit of Existing Tilt Up Concrete Wall



Loads
Distributed user loads - Members

Wind Loads Acting on Retrofit of Existing Tilt Up Concrete Wall with HSS Stiff Back



Deflections of Retrofit of Existing Tilt Up Concrete Wall



RAM Elements **Salas O'Brien**

Current Date: 8/17/2026 5:41 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 WALLS\04.2 Existing Wall\New Overhead Roll Up Door\New Strong Back to Support Existing Wall from Out of Plane Wind_REV 1.retx

Steel Code Check
Comprehensive Report

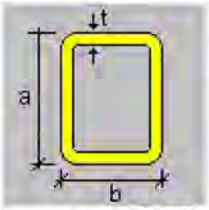
AISC 360-2016 LRFD (Hot-rolled)

Member : 1 (Wall Opening Stiff Back) - OK

Section information

Section name: HSS_RECT 12X8X3_8 **(US)**

Dimensions



a	=	12.000	[in]	Height
b	=	8.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	13.200	
Moment of Inertia (local axes) (I)	[in4]	262.000	140.000
Moment of Inertia (principal axes) (I')	[in4]	262.000	140.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.455	3.257
Radius of gyration (principal axes) (r')	[in]	4.455	3.257
Saint-Venant torsion constant. (J)	[in4]	293.000	
Section warping constant. (Cw)	[in6]	97.043	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	43.700	35.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	43.700	35.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	43.700	35.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	43.700	35.100
Plastic section modulus (local axis) (Z)	[in3]	53.000	40.100
Plastic section modulus (principal axis) (Z')	[in3]	53.000	40.100
Polar radius of gyration. (ro)	[in]	5.533	
Area for shear (Aw)	[in2]	4.853	7.645
Torsional constant. (C)	[in3]	62.053	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	43.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
43.00	43.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
43.00	43.00	43.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	594.00 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	594.00	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	660.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.01			
Capacity	:	222.30 [Kip]	Reference	:	Cl.E3
Demand	:	2.69 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	19.92	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.38	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	222.30	Cl.E3
Effective length (KL_{33})	[ft]	43.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	115.82	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	21.34	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	13.20	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	18.71	Eq.E3-3
Nominal flexural buckling strength (P_{n33})	[Kip]	247.00	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.02			
Capacity	:	118.79 [Kip]	Reference	:	Cl.E3
Demand	:	2.69 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	19.92	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	31.38	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	118.79	Cl.E3
Effective length (KL_{22})	[ft]	43.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	158.44	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	11.40	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	13.20	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	10.00	Eq.E3-3
Nominal flexural buckling strength (P_{n22})	[Kip]	131.99	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.35			
Capacity	:	198.75 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	69.63 [Kip*ft]	Ctrl Eq.	:	D3 at 49.85%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	19.92	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	

Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	31.38	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	198.75	Cl.F7.1
Yielding (M_n)	[Kip*ft]	220.83	Eq.F7-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	198.75	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	24.01	Eq.F7-12
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	640.02	Eq.F7-13
Lateral-torsional buckling modification factor (C_b)	–	1.14	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	220.83	Eq.F7-10

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	138.15 [Kip*ft]	Reference	: Cl.F7.2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	–	Compact	
Unstiffened element slenderness (λ)	–	19.92	
Limiting slenderness for noncompact unstiffened element (λ_r)	–	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	–	26.97	
Stiffened element classification	–	Compact	
Stiffened element slenderness (λ)	–	31.38	
Limiting slenderness for noncompact stiffened element (λ_r)	–	137.27	
Limiting slenderness for compact stiffened element (λ_p)	–	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	150.38	Cl.F7.1
Yielding (M_n)	[Kip*ft]	167.08	Eq.F7-1
Factored web local buckling strength about a geometric axis(ϕM_n):	[Kip*ft]	138.15	Cl.F7.2
Local web buckling (M_n)	[Kip*ft]	153.50	Eq.F7-2

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	131.04 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D3 at 46.51%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	131.04	Cl.G1
Web buckling coefficient (k_v)	–	5.00	Cl.G4
Web buckling coefficient (C_v)	–	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	145.60	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.03		
Capacity	:	206.42 [Kip]	Reference	: Cl.G1
Demand	:	6.46 [Kip]	Ctrl Eq.	: D3 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	206.42	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	229.36	Eq.G4-1

Torsion Design ✓

Torsion

Ratio	:	0.00			
Capacity	:	139.62 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	139.62	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	155.13	Eq.H3-1

Combined Actions Design ✓

Combined flexure and axial

Ratio	:	0.36			
Ctrl Eq.	:	D3 at 49.85%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.36	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	198.75	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	138.15	Cl.H1.1
Available axial strength (P_c)	[Kip]	118.79	Cl.H1.1

Post Installed Anchors for Stiff Back at New Overhead Door

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 23, 2026	Date:	5/23/2026
Fastening point:			

Specifier's comments:

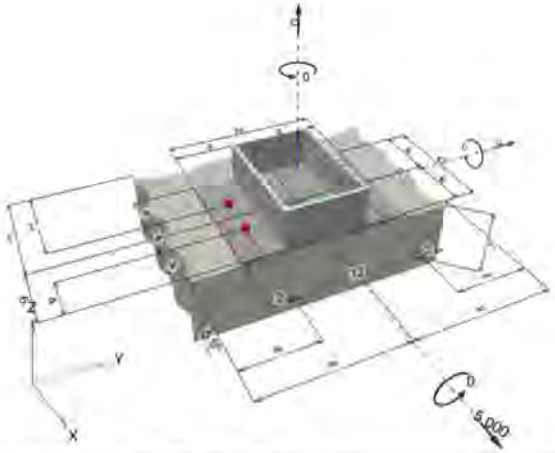
1 Input data



Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	HILTI ⌀ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R]:	l_x x l_y x t = 12.000 in. x 16.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS12X8X.375; (L x W x T) = 12.000 in. x 8.000 in. x 0.375 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 7.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



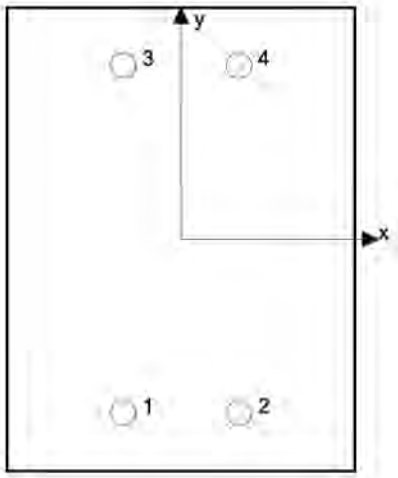
Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 5,000; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	98

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,250	1,250	0
2	0	1,250	1,250	0
3	0	1,250	1,250	0
4	0	1,250	1,250	0

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,250	9,996	13	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	5,000	21,018	24	OK
Concrete edge failure in direction x+**	5,000	5,136	98	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.39	81,600

Calculations

V_{sa} [lb]
16,660

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
16,660	0.600	9,996	1,250

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Concrete - May 23, 2026	Date:	5/23/2026
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	0.000	0.000	4.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.690	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
308.88	127.92	1.000	1.000	0.912	1.000	6,816

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
30,026	0.700	21,018	5,000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
4.000	-	0.000	1.000	7.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
3.770	1.000	0.750	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [lb]
144.00	72.00	1.000	1.000	1.000	3,669

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
7,338	0.700	5,136	5,000

*Anchor row defined by: Anchor 2, 4; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Design:	Concrete - May 23, 2026	Date:	5/23/2026
Fastening point:			

6 Installation data

Profile: Rectangular HSS (AISC), HSS12X8X.375; (L x W x T) = 12.000 in. x 8.000 in. x 0.375 in.

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)

Item number: 418084 KH-EZ 3/4"x5 1/2"

Maximum installation torque: 1,140 in.lb

Hole diameter in the base material: 0.750 in.

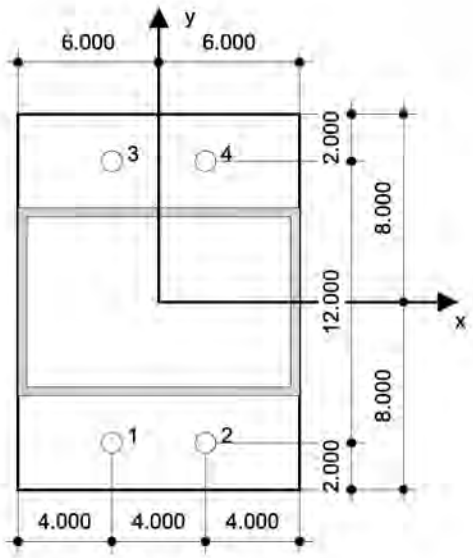
Hole depth in the base material: 5.375 in.

Minimum thickness of the base material: 7.000 in.

Hilti ⌀ 3/4 in KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 5 in nominal embedment depth per ICC-ES ESR-3027 , Hammer drill bit installation per MPII

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench • Hilti SIW 9-A22 Impact Wrench



Coordinates Anchor [in.]

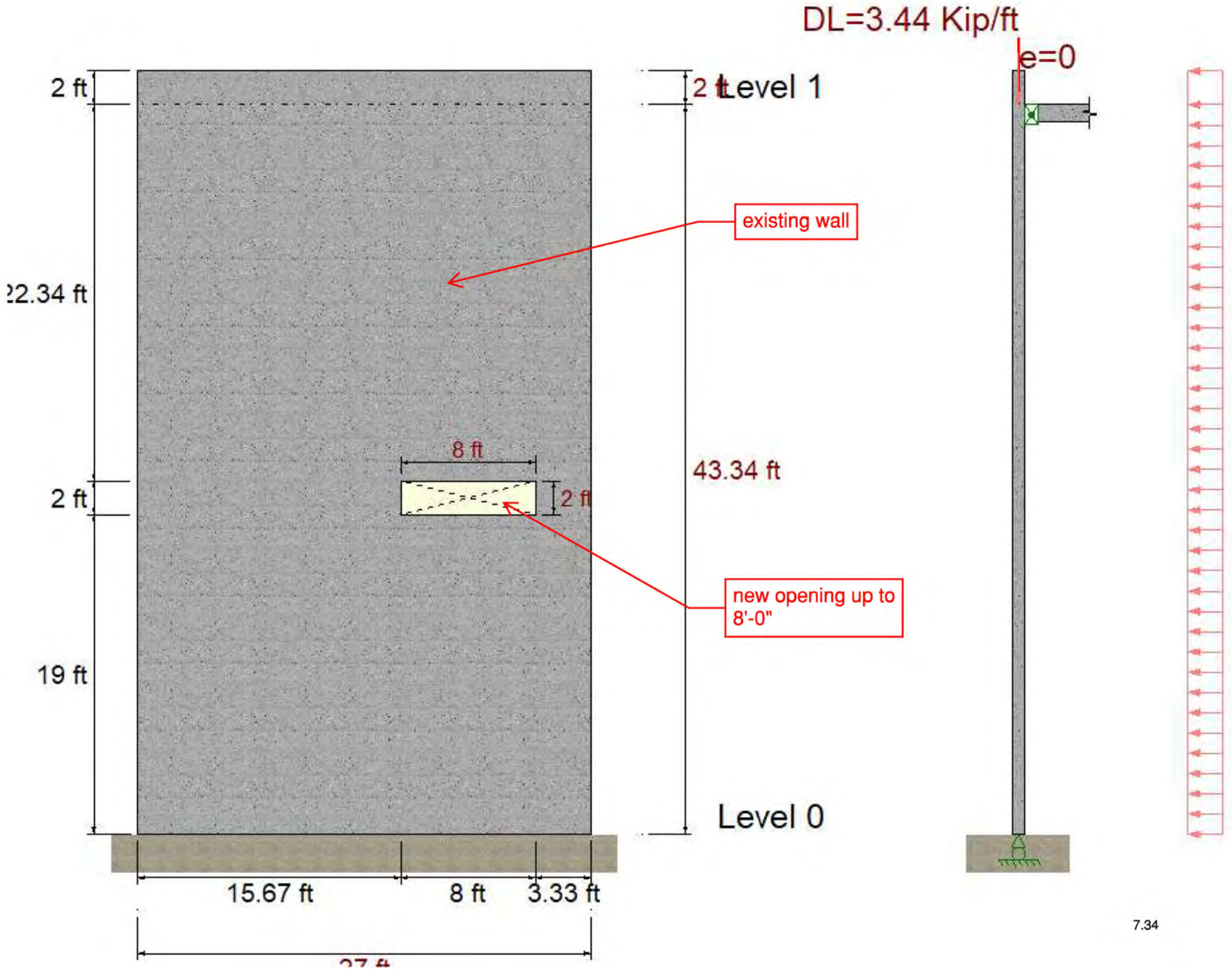
Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.000	-6.000	-	8.000	-	-
2	2.000	-6.000	-	4.000	-	-
3	-2.000	6.000	-	8.000	-	-
4	2.000	6.000	-	4.000	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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DL=3.44 Kip/ft
e=0
2 ft
Level 1
existing wall
22.34 ft
8 ft
2 ft
2 ft
43.34 ft
new opening up to 8'-0"
19 ft
Level 0
15.67 ft
8 ft
3.33 ft

Load Development: New Openings in Existing Wall

Unit weight of concrete = 145 pcf
Wall thickness = 9 1/4"
Wall height above opening = 26'-0"

Existing joists @ 6'-0" OC
Existing roof joist DL = 127 plf (end reaction = (127*50ft/2) = 3.18 kip, 0.53 kip/ft)
Existing roof joist LLr = 108 plf (end reaction = (108*50ft/2) = 2.70 kip, 0.45 kip/ft)

Self weight of wall = (145 pcf)*(9.25 in*1ft/12in)*(26 ft) = 2.91 kip/ft

Imposed Loads:
DL = 3.44 kip/ft
LLr = 2.7 kip/ft

Max Load Combination = 8.45 kip/ft

Capacity of 3/4" DIA through bolt @ 8" OC = 52.5 kip/ft ---------> 8.45/52.5 = 0.16 OKAY

Reaction at ends = 33.94 kip

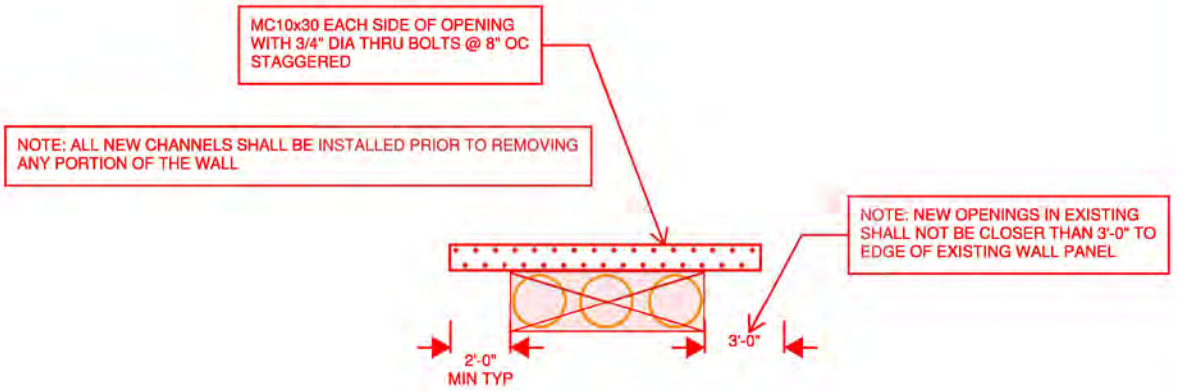
Developed length at ends = 2'-0"

Through bolt capacity at end bearing = 104.00 kip --------> 33.94/104.00 = 0.33 OKAY

f'c = 4000 psi

Force per bolt = 6 kip

Bolt bearing = (6 kip)/(6.94 in^2) = 816 psi --------> OKAY



RAM Elements **Salas O'Brien**

Current Date: 5/28/2026 3:12 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\04 Walls\04.2 Existing Wall\Typ Existing Tilt Up Panel.cwd

Design Results
Concrete Wall

General Information

Global status : OK

Design code	:	ACI 318-19

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	45.34 [ft]
Total length	:	27.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	43.34	9.25

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	15.67	19.00	8.00	2.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LLr	No	LLR	Roof Live Load
WL	No	WIND	Wind Load
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5LLr
D3	Yes		1.2DL+1.6LLr
D4	Yes		1.2DL+0.5WL
D5	Yes		1.2DL+1.6LLr+0.5WL
D6	Yes		1.2DL+WL
D7	Yes		1.2DL+WL+0.5LLr
D8	Yes		0.9DL+WL

Loads

Distributed loads:

Consider self weight : DL

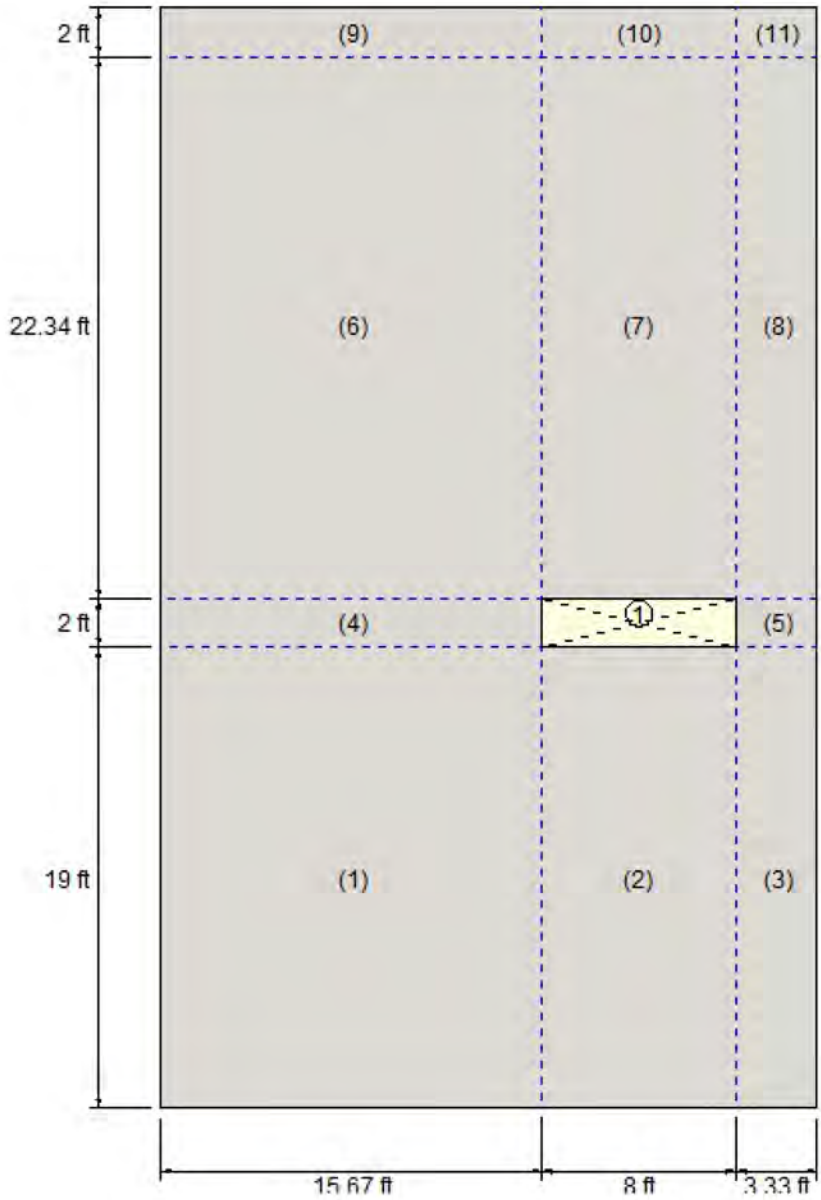
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	DL	Vertical	3.44	0.00
1	LLr	Vertical	2.70	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	WL	0.04

Bearing Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	15.67	19.00
2	15.67	0.00	8.00	19.00
3	23.67	0.00	3.33	19.00
4	0.00	19.00	15.67	2.00
5	23.67	19.00	3.33	2.00
6	0.00	21.00	15.67	22.34
7	15.67	21.00	8.00	22.34
8	23.67	21.00	3.33	22.34
9	0.00	43.34	15.67	2.00
10	15.67	43.34	8.00	2.00
11	23.67	43.34	3.33	2.00

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	11-#6	18.00	28.46
2	6-#5	18.00	23.72
3	4-#6	13.00	28.46
4	11-#6	18.00	28.46
5	4-#6	13.00	28.46
6	11-#6	18.00	28.46
7	6-#6	18.00	28.46
8	4-#6	13.00	28.46
9	11-#5	18.00	23.72
10	6-#5	18.00	23.72
11	3-#5	18.00	23.72

Intermediate results for axial-bending

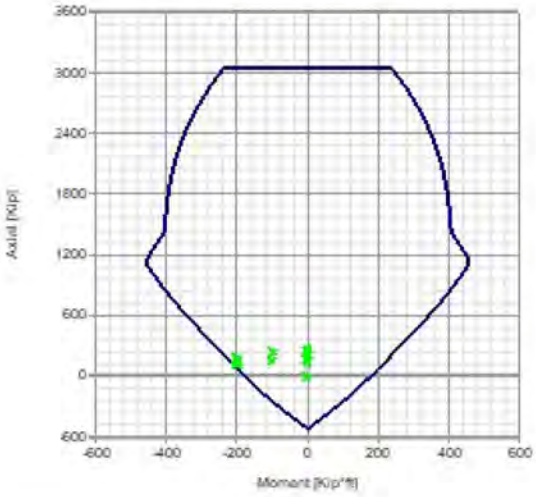
Segment	Condition	c [in]	d [in]
1	D8 (Max)	1.12	7.63
2	D8 (Max)	0.91	7.69
3	D8 (Top)	1.43	7.63
4	D8 (Max)	1.12	7.63
5	D8 (Top)	1.47	7.63
6	D8 (Max)	1.11	7.63
7	D8 (Max)	1.12	7.63
8	D8 (Bottom)	1.43	7.63
9	D8 (Max)	0.76	7.69
10	D8 (Max)	0.80	7.69
11	D8 (Max)	0.91	7.69

Combined axial flexure

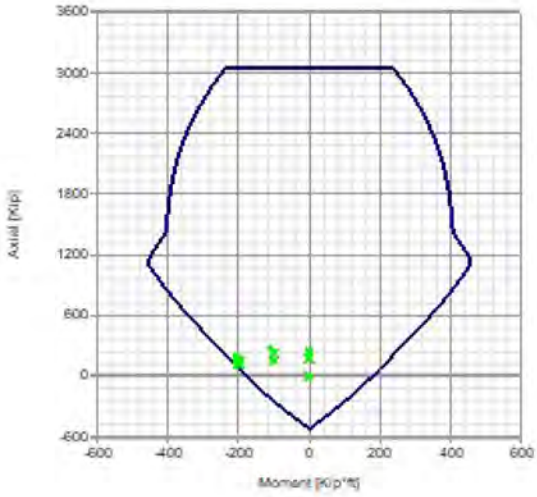
Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D8 (Max)	114.27	-199.13	211.57	0.94
2	D8 (Max)	41.45	-72.36	83.94	0.86
3	D8 (Top)	33.86	-54.16	68.53	0.79
4	D8 (Max)	110.92	-199.95	210.65	0.95
5	D8 (Top)	40.96	-64.09	70.42	0.91
6	D8 (Max)	102.99	-191.66	208.45	0.92
7	D8 (Max)	40.28	-86.12	107.99	0.80
8	D8 (Bottom)	33.27	-55.21	68.38	0.81
9	D8 (Max)	3.60	0.65	133.20	0.00
10	D8 (Max)	2.06	-0.44	72.17	0.01
11	D8 (Max)	0.74	2.24	34.98	0.06

Interaction diagrams, P vs. M

P vs. M (Segment 1)



P vs. M (Segment 4)



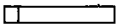
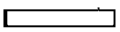
Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D3 (Bottom)	252.64	3058.09	0.08
2	D5 (Bottom)	137.56	1563.41	0.09
3	D5 (Max)	70.80	647.28	0.11
4	D5 (Bottom)	237.35	3058.09	0.08
5	D5 (Bottom)	87.06	647.28	0.13
6	D5 (Bottom)	216.38	3058.09	0.07
7	D5 (Bottom)	89.16	1560.65	0.06
8	D3 (Bottom)	69.95	647.28	0.11
9	DC1 (Bottom)	5.61	3063.15	0.00
10	D1 (Bottom)	3.21	1563.41	0.00
11	D1 (Bottom)	1.15	650.22	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	522.72	0.00
2	DC1 (Top)	0.00	200.88	0.00
3	DC1 (Top)	0.00	190.08	0.00
4	DC1 (Top)	0.00	522.72	0.00
5	DC1 (Top)	0.00	190.08	0.00
6	DC1 (Top)	0.00	522.72	0.00
7	DC1 (Top)	0.00	285.12	0.00
8	DC1 (Top)	0.00	190.08	0.00
9	DC1 (Top)	0.00	368.28	0.00
10	DC1 (Top)	0.00	200.88	0.00
11	DC1 (Top)	0.00	100.44	0.00

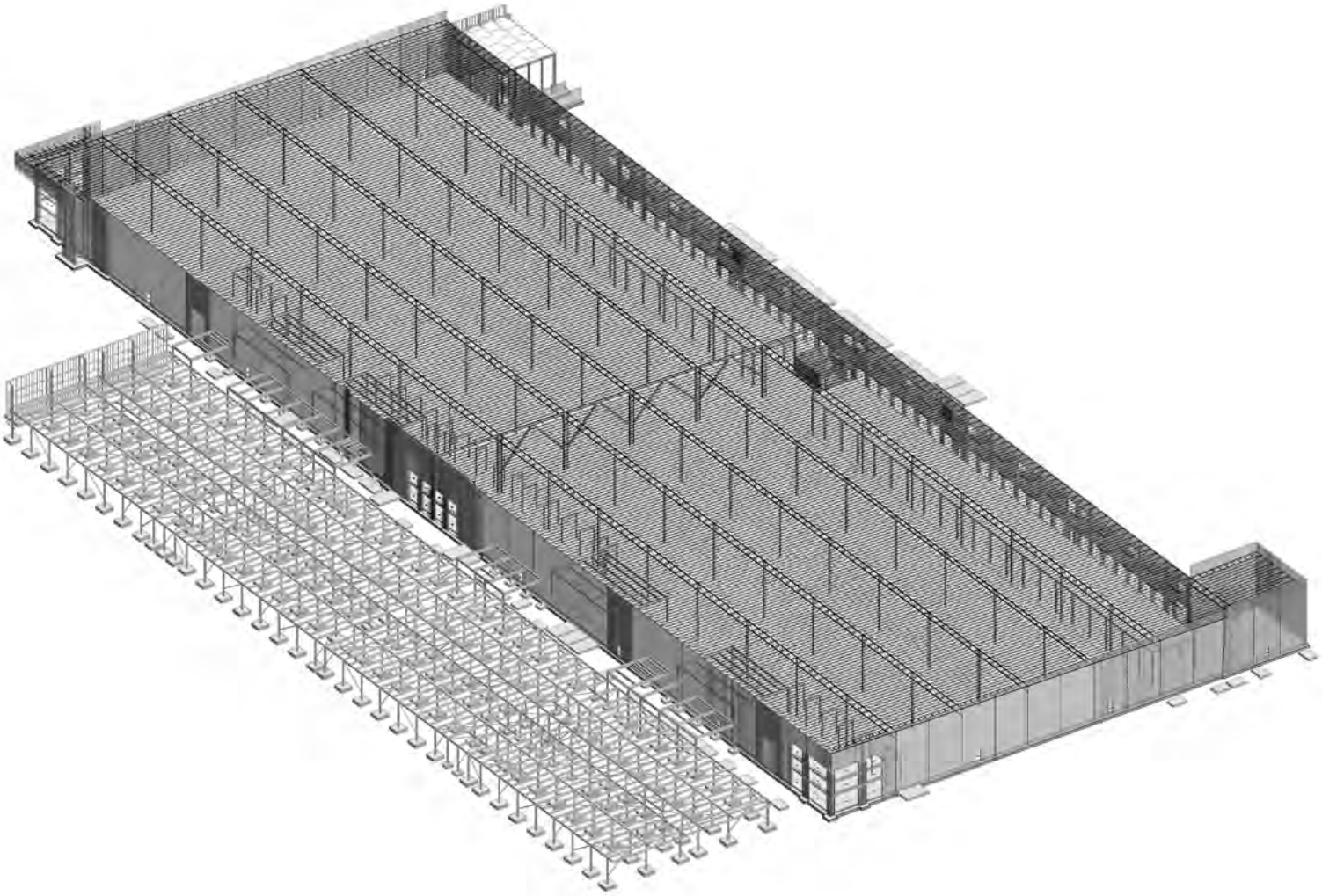
Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn	
1	D8 (Bottom)	16.130	116.624	0.14	
2	D8 (Bottom)	8.575	55.569	0.15	
3	D8 (Top)	9.086	29.627	0.31	
4	D8 (Bottom)	1.948	114.598	0.02	
5	D8 (Bottom)	0.724	30.430	0.02	
6	D8 (Max)	9.328	113.445	0.08	
7	D8 (Max)	7.101	57.821	0.12	
8	D8 (Bottom)	9.934	29.566	0.34	
9	D8 (Bottom)	1.072	92.377	0.01	
10	D8 (Bottom)	0.014	48.232	0.00	
11	D8 (Bottom)	1.366	21.323	0.06	

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Chapter 8 - Pump House Design



Chavez-Grieves
consulting engineers, inc.

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PHONE: (505) 344-4080 FAX: (505) 343-8759

SHEET NO. 1 OF 1
PROJECT
SUBJECT Roof Loads
CLIENT JOB NO.
CALCULATED BY DATE 8/16/2017
CHECKED BY DATE

ROOF LOADS

ROOF DEAD LOADS

				LOADS
Deck:	Metal Decking, 1 1/2" wide rib (type B), 20 ga			2.3 psf
	SELF WEIGHT INCLUDED IN RAMSS? NO			
Insulation:	Rolled or Batt, Rock Wool	6.0 in Thk		1.2 psf
		0.2 psf / inch of thickness		
Roofing Materials:	Standing Seam Metal Roofing			3.0 psf
Topping/Flute Infill:	N/A	0.000 in Thk		0.0 psf
		0.0 psf / inch of thickness		
Joists/Beams:	Wide Flange Framing (Self Weight in RamSS)			0.0 psf
Mech/Elec/Plumb/Lights:	MINIMUM for any application			4.0 psf
Ceiling:	None			0.0 psf
Sprinklers:	No			0.0 psf
Other:				psf
Misc:				psf
PV System:	None			0.0 psf

ROOF LIVE LOADS

Live Load: (IBC 2021 Table 1607.1) 20.0 psf

ROOF LOADS SUMMARY

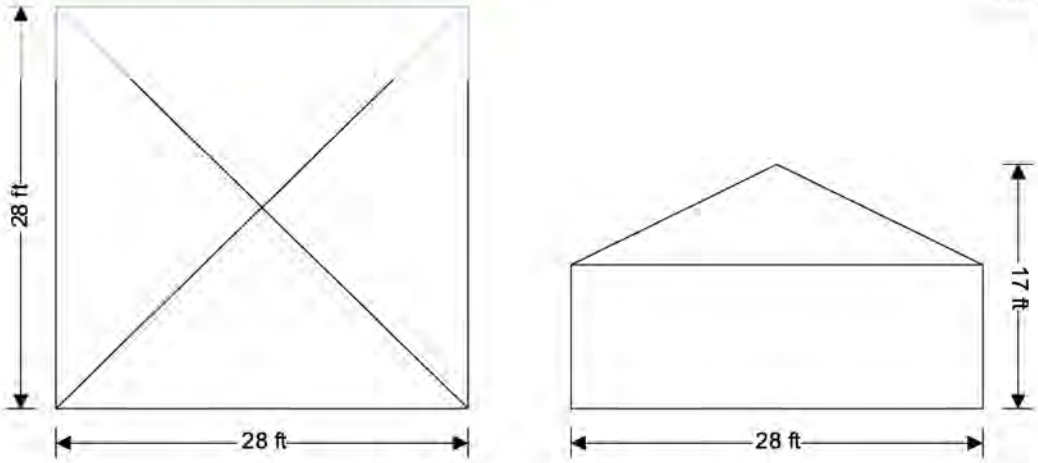
Dead Load =	10.5	psf	User Note: Verify that RAM SS is set to the appropriate selfweight
Construction Dead Load =	0.0	psf	User Note: CDL does not apply to non-composite beams
Roof Live Load =	20.0	psf	User Note: Snow Loads need to be checked separately
Partition =	0.0	psf	
Construction Live Load =	0.0	psf	User Note: CLL does not apply to non-composite beams
Mass Dead Load =	10.5	psf	Use 12 PSF top Chord dead load

WIND LOADING

In accordance with ASCE7-22

Using the components and cladding design method

Tedds calculation version 2.1.19



Building data

Type of roof	Hipped
Length of building	b = **28.00** ft
Width of building	d = **28.00** ft
Height to eaves	H = **10.00** ft
Pitch of main slope	α_0 = **26.6** deg
Pitch of gable slope	α_{90} = **26.6** deg
Mean height	h = **13.50** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.00** ft

General wind load requirements

Basic wind speed	V = **140.0** mph
Risk category	II
Wind directionality factor (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**

Velocity pressure $q_h = 0.00256 \times K_z \times K_{zt} \times K_e \times V^2 \times 1\text{psf/mph}^2 =$ **42.6** psf

Peak velocity pressure for internal pressure

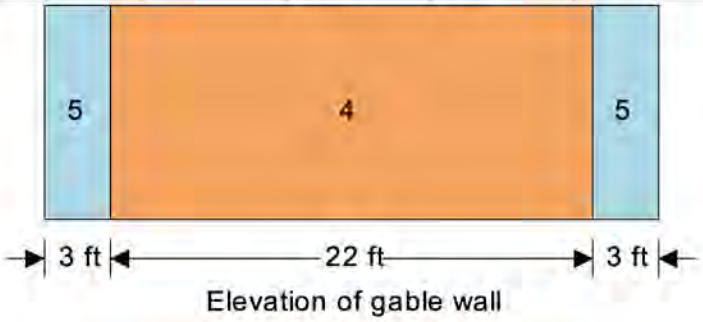
Peak velocity pressure – internal (as roof press.) $q_i =$ **42.65** psf

Equations used in tables

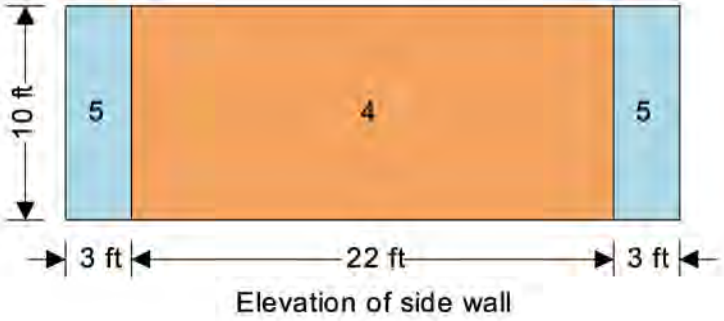
Net pressure $p = K_d \times (q_h \times (GC_p - GC_{pi}))$

Components and cladding pressures - Wall (Table 30.3-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	1.00	-1.10	42.8	-46.4
50 sf	4	-	-	50.0	0.88	-0.98	38.3	-41.9
200 sf	4	-	-	200.0	0.77	-0.87	34.4	-38.1
>500 sf	4	-	-	500.1	0.70	-0.80	31.9	-35.5
<=10 sf	5	-	-	10.0	1.00	-1.40	42.8	-57.3
50 sf	5	-	-	50.0	0.88	-1.15	38.3	-48.3
200 sf	5	-	-	200.0	0.77	-0.94	34.4	-40.6
>500 sf	5	-	-	500.1	0.70	-0.80	31.9	-35.5



Elevation of gable wall

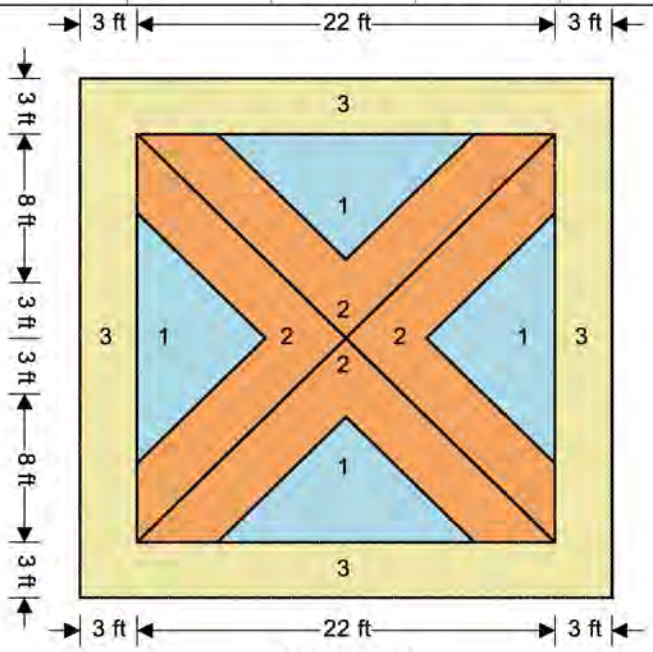


Elevation of side wall

Components and cladding pressures - Roof (Figure Figure 30.3-2F)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.70	-1.40	31.9	-57.3
100 sf	1	-	-	100.0	0.30	-0.80	17.4	-35.5
200 sf	1	-	-	200.0	0.30	-0.80	17.4	-35.5

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
>300 sf	1	-	-	300.1	0.30	-0.80	17.4	-35.5
<=10 sf	2	-	-	10.0	0.70	-2.00	31.9	-79.0
100 sf	2	-	-	100.0	0.30	-1.00	17.4	-42.8
200 sf	2	-	-	200.0	0.30	-1.00	17.4	-42.8
>300 sf	2	-	-	300.1	0.30	-1.00	17.4	-42.8
<=10 sf	3	-	-	10.0	0.70	-2.00	31.9	-79.0
100 sf	3	-	-	100.0	0.30	-1.23	17.4	-51.2
200 sf	3	-	-	200.0	0.30	-1.00	17.4	-42.8
>300 sf	3	-	-	300.1	0.30	-1.00	17.4	-42.8



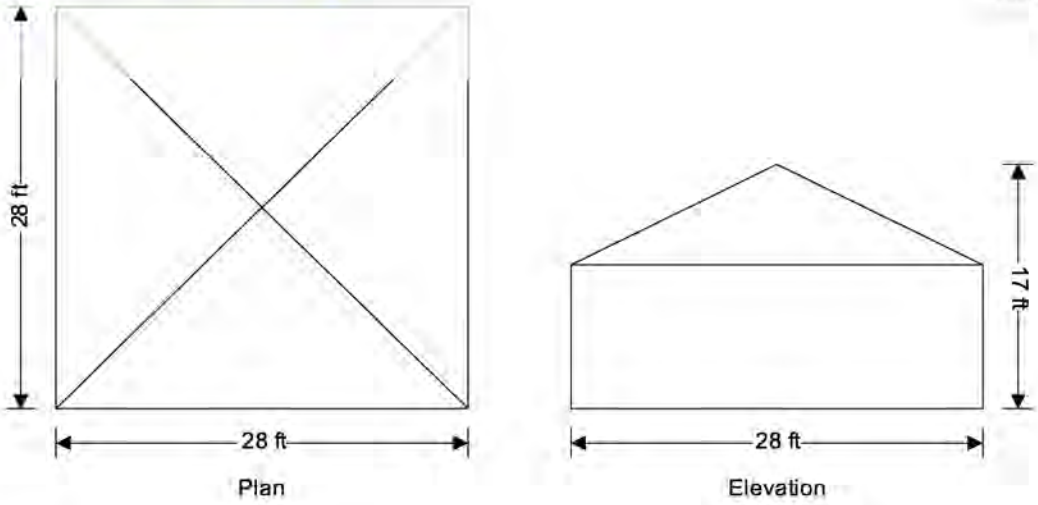
Plan on roof

WIND LOADING

In accordance with ASCE7-22

Using the directional design method

Tedds calculation version 2.1.19



Building data

Type of roof	Hipped
Length of building	b = **28.00** ft
Width of building	d = **28.00** ft
Height to eaves	H = **10.00** ft
Pitch of main slope	α_0 = **26.6** deg
Pitch of gable slope	α_{90} = **26.6** deg
Mean height	h = **13.50** ft

General wind load requirements

Basic wind speed	V = **140.0** mph
Risk category	II
Wind directionality factor (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum roof design wind load (cl.27.1.5)	p_{min_r} = **8** psf
Minimum wall design wind load (cl.27.1.5)	p_{min_w} = **16** psf

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_e \times V^2 \times 1psf/mph^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
10.00	0.85	42.65
13.50	0.85	42.65

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **42.65** psf

Pressures and forces

Net pressure $p = K_d \times q \times G_f \times C_{pe} - K_d \times q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.50	-0.26	42.65	-14.65	219.14	-3.21
B (-ve)	13.50	-0.60	42.65	-25.01	219.14	-5.48
C (-ve)	13.50	-0.90	42.65	-34.26	50.95	-1.75
D (-ve)	13.50	-0.90	42.65	-34.26	152.84	-5.24
E (-ve)	13.50	-0.50	42.65	-21.93	233.38	-5.12
F (-ve)	13.50	-0.30	42.65	-15.77	1.11	-0.02

Total vertical net force $F_{w,v}$ = **-18.61** kips

Total horizontal net force $F_{w,h}$ = **1.02** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	42.65	18.13	280.00	5.08
B	13.50	-0.50	42.65	-21.93	280.00	-6.14
C	13.50	-0.70	42.65	-28.10	280.00	-7.87
D	13.50	-0.70	42.65	-28.10	280.00	-7.87

Overall loading

Projected vertical plan area of wall $A_{vert_w_0} = b \times H$ = **280.00** ft^2

Projected vertical area of roof $A_{vert_r_0} = b \times d/2 \times \tan(\alpha_0) - (d/2 \times \tan(\alpha_0))^2 / \tan(\alpha_{90})$ = **98.02** ft^2

Minimum overall horizontal loading $F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0}$ = **5.26** kips

Leeward net force $F_l = F_{w,wB}$ = **-6.1** kips

Windward net force $F_w = F_{w,wA}$ = **5.1** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min})$ = **12.2** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $+C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.50	0.21	42.65	12.91	219.14	2.83

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
B (+ve)	13.50	-0.60	42.65	-11.96	219.14	-2.62
C (+ve)	13.50	-0.18	42.65	0.98	50.95	0.05
D (+ve)	13.50	-0.18	42.65	0.98	152.84	0.15
E (+ve)	13.50	-0.18	42.65	0.98	233.38	0.23
F (+ve)	13.50	-0.18	42.65	0.98	1.11	0.00

Total vertical net force $F_{w,v}$ = **0.57** kips

Total horizontal net force $F_{w,h}$ = **2.44** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	42.65	31.18	280.00	8.73
B	13.50	-0.50	42.65	-8.88	280.00	-2.49
C	13.50	-0.70	42.65	-15.04	280.00	-4.21
D	13.50	-0.70	42.65	-15.04	280.00	-4.21

Overall loading

Projected vertical plan area of wall $A_{vert_w_0} = b \times H$ = **280.00** ft^2

Projected vertical area of roof $A_{vert_r_0} = b \times d/2 \times \tan(\alpha_0) - (d/2 \times \tan(\alpha_0))^2 / \tan(\alpha_{90})$ = **98.02** ft^2

Minimum overall horizontal loading $F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0}$ = **5.26** kips

Leeward net force $F_l = F_{w,wB}$ = **-2.5** kips

Windward net force $F_w = F_{w,wA}$ = **8.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min})$ = **13.7** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.50	-0.26	42.65	-14.65	219.14	-3.21
B (-ve)	13.50	-0.60	42.65	-25.01	219.14	-5.48
C (-ve)	13.50	-0.90	42.65	-34.26	50.95	-1.75
D (-ve)	13.50	-0.90	42.65	-34.26	152.84	-5.24
E (-ve)	13.50	-0.50	42.65	-21.93	233.38	-5.12
F (-ve)	13.50	-0.30	42.65	-15.77	1.11	-0.02

Total vertical net force $F_{w,v}$ = **-18.61** kips

Total horizontal net force $F_{w,h}$ = **1.02** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	42.65	18.13	280.00	5.08
B	13.50	-0.50	42.65	-21.93	280.00	-6.14
C	13.50	-0.70	42.65	-28.10	280.00	-7.87
D	13.50	-0.70	42.65	-28.10	280.00	-7.87

Overall loading

Projected vertical plan area of wall $A_{vert_w_90} = d \times H = \mathbf{280.00}$ ft^2

Projected vertical area of roof $A_{vert_r_90} = d^2/4 \times \tan(\alpha_0) = \mathbf{98.02}$ ft^2

Minimum overall horizontal loading $F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = \mathbf{5.26}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-6.1}$ kips

Windward net force $F_w = F_{w,wA} = \mathbf{5.1}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{12.2}$ kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.50	0.21	42.65	12.91	219.14	2.83
B (+ve)	13.50	-0.60	42.65	-11.96	219.14	-2.62
C (+ve)	13.50	-0.18	42.65	0.98	50.95	0.05
D (+ve)	13.50	-0.18	42.65	0.98	152.84	0.15
E (+ve)	13.50	-0.18	42.65	0.98	233.38	0.23
F (+ve)	13.50	-0.18	42.65	0.98	1.11	0.00

Total vertical net force $F_{w,v} = \mathbf{0.57}$ kips

Total horizontal net force $F_{w,h} = \mathbf{2.44}$ kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	10.00	0.80	42.65	31.18	280.00	8.73
B	13.50	-0.50	42.65	-8.88	280.00	-2.49
C	13.50	-0.70	42.65	-15.04	280.00	-4.21
D	13.50	-0.70	42.65	-15.04	280.00	-4.21

Overall loading

Projected vertical plan area of wall $A_{vert_w_90} = d \times H = \mathbf{280.00}$ ft^2

Projected vertical area of roof $A_{vert_r_90} = d^2/4 \times \tan(\alpha_0) = \mathbf{98.02}$ ft^2

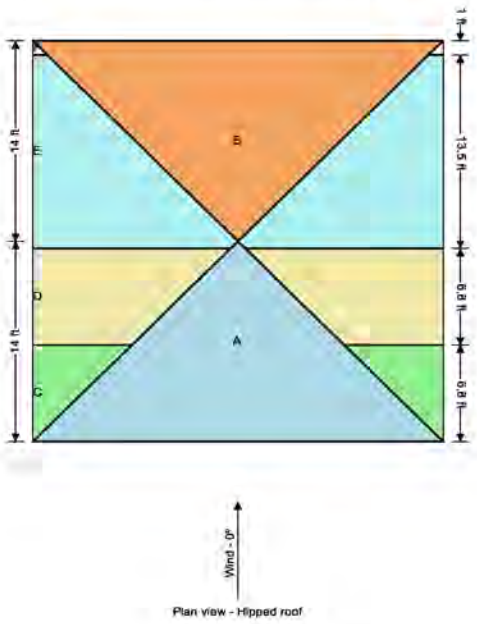
Minimum overall horizontal loading $F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = \mathbf{5.26}$ kips

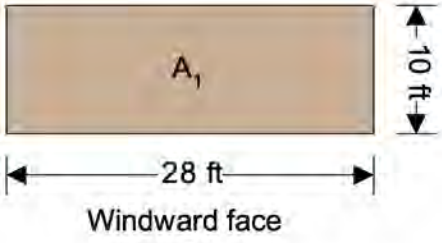
Leeward net force $F_l = F_{w,wB} = \mathbf{-2.5}$ kips

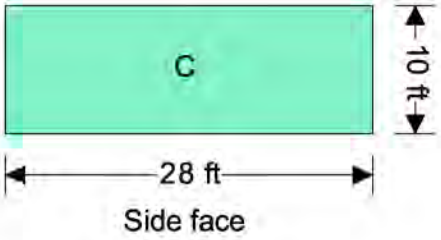
Windward net force $F_w = F_{w,wA} = \mathbf{8.7}$ kips

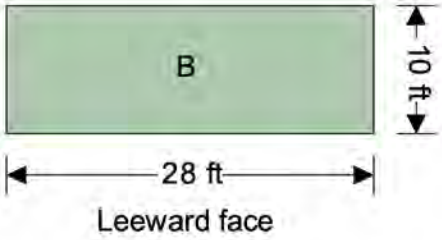
Overall horizontal loading

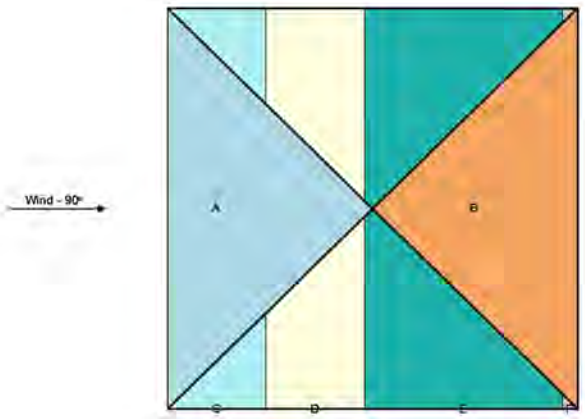
$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{13.7}$ kips



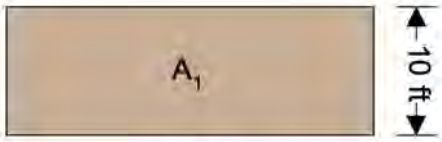








Plan view - Hipped roof



Windward face



Side face



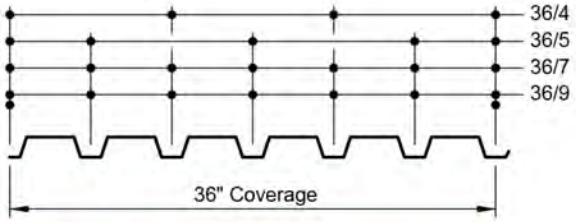
Leeward face

PUMP HOUSE - DIAPHRAGM DESIGN

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User Defined Criteria	
Deck Application:	Roof
Deck Type:	B
Gage:	22
Yield Stress (ksi):	40
Net Uplift (psf):	25
Support Fasteners:	Arc Spot Welds Visible Diameter: 0.625
Side Lap Fasteners:	Screws #10



SUMMARY:

36/4
SUPPORT SPACING = 5 FT
NO. SIDELAP CONNECTORS = 1
ALLOWABLE SHEAR = 250 PLF

MAX LATERAL LOAD = 120 PLF
*FROM *LOADING AND CRITERIA - CMU WALL*

B, BV 22 GA. Diaphragm Design

The values shown in the tables are nominal strengths and are not to be used without applying the proper safety or resistance factor as shown above the top-right-hand corner of table. The factors are to be applied as follows:

- **LRFD** – The table values must be multiplied by the Φ resistance factor when comparing to forces calculated using Load and Resistance Factor Design.
- **ASD** – The table values must be divided by the Ω safety factor when comparing to forces calculated using Allowable Strength Design.

When diaphragm design includes net uplift only ASD Wind and Buckling safety factors are provided.

The shear strength values indicated are calculated based on the number of side lap connections being equally spaced per span. When maximum spacing of side lap connections is preferred in lieu of number equally spaced per span, due to the fasteners being placed at one-half space from each support with whole spaces between, divide span by required fastener spacing and round up to nearest integer. Use values in the table row corresponding to calculated number of fasteners per span.

Average spacing of support connections parallel to deck flutes is assumed equal to the side lap connection spacing. It may be possible to achieve greater diaphragm shear strengths by decreasing the spacing of support connections parallel to deck flutes. Please contact New Millennium for details.

The shear strength fields shown as blank (-) are conditions that do not meet minimum Steel Deck Institute side lap connection requirements. The values shown in RED indicate conditions with 0 side lap fasteners not in compliance with minimum Steel Deck Institute requirements. These values can be used to determine diaphragm shear strengths when properly spaced side lap connections are ignored for conservatism as part of the diaphragm shear design.

Designs for bare deck (no concrete fill) show nominal diaphragm shear values due to buckling in table below the shear strength table, for use in determining when conditions may be limited by panel buckling. An asterisk (*) is shown following the value in the shear strength table indicating conditions where panel buckling could potentially govern over connector strength. Nominal shear strength values flagged in this manner denote that a factored shear strength value exceeds the factored buckling shear strength value. The Designer should compare the factored values to determine which design requirement is governing.

See Diaphragm table on page 2

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B, BV 22 GA. Diaphragm Design

Fy = 40 ksi, Fu = 50 ksi, Fxx = 60 ksi, Design Thickness = 0.0295 in.
Support Fasteners: 5/8" Arc Spot Welds
Side Lap Fasteners: #10 Screws

Ω (Wind): 2.15
Ω (Buckling): 2.00

Support Fastener Pattern	Side Lap Conn./ Span	Bare Deck Nominal Shear Strength (plf) in Presence of 25 psf Net Uplift														K_1 (ft^{-1})
		Deck Span (ft - in)														
		3 - 0	3 - 6	4 - 0	4 - 6	5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	7 - 6	8 - 0	8 - 6	9 - 0	9 - 6	
36/9 Dn = 91	0	1941	1707	1521	1350	1206	1088	990	907	836	774	721	673	635	602	0.324
	1	2051	1810	1615	1453	1299	1173	1068	-	-	-	-	-	-	-	0.272
	2	2156	1909	1708	1542	1392	1257	1145	1050	969	898	836	782	738	-	0.234
	3	2257	2005	1798	1626	1483	1341	1222	1121	1035	960	894	837	790	748	0.206
	4	2353	2097	1885	1708	1560	1426	1300	1193	1101	1022	952	891	841	797	0.183
	5	2445	2186	1970	1788	1635	1504	1377	1264	1167	1084	1010	946	893	846	0.165
	6	2533	2272	2052	1866	1709	1574	1454	1335	1234	1146	1068	1000	944	894	0.151
	7	2616	2354	2131	1942	1781	1642	1522	1407	1300	1207	1126	1055	996	943	0.138
	8	2696	2433	2208	2016	1851	1709	1585	1477	1366	1269	1184	1109	1047	992	0.128
	9	2771	2509	2282	2088	1920	1774	1648	1537	1432	1331	1242	1164	1099	1041	0.119
	10	2843	2581	2354	2157	1987	1839	1709	1595	1495	1393	1300	1219	1150	1090	0.111
36/7 Dn = 91	0	1242	1080	941	831	744	672	612	562	519	481	448	419	352	330	0.486
	1	1370	1195	1057	934	837	757	690	-	-	-	-	-	-	-	0.377
	2	1493	1306	1159	1038	929	841	767	705	651	605	564	528	455	-	0.308
	3	1610	1414	1258	1131	1022	925	844	776	717	667	622	583	506	476	0.261
	4	1722	1518	1354	1220	1109	1009	922	847	784	728	680	638	558	525	0.226
	5	1829	1618	1447	1306	1189	1090	999	919	850	790	738	692	610	574	0.199
	6	1931	1715	1537	1391	1268	1164	1075	990	916	852	796	747	661	623	0.178
	7	2027	1807	1625	1473	1345	1236	1143	1061	982	914	854	801	713	671	0.161
	8	2118	1895	1709	1552	1420	1306	1209	1124	1049	976	912	856	764	720	0.147
	9	2204	1979	1790	1629	1493	1375	1274	1186	1109	1038	970	910	816	769	0.135
	10	2285	2060	1868	1704	1564	1443	1338	1247	1166	1095	1028	965	867	818	0.125
36/5 Dn = 572	0	1099	967	861	769	688	621	500	451	408	371	338	309	285	263	0.583
	1	1208	1068	955	862	780	705	578	-	-	-	-	-	-	-	0.433
	2	1309	1164	1045	946	863	790	655	594	541	495	454	418	388	-	0.345
	3	1402	1254	1130	1026	938	863	729	665	607	557	512	473	439	409	0.286
	4	1488	1338	1211	1103	1011	932	794	731	673	618	570	527	491	458	0.245
	5	1566	1416	1287	1177	1081	999	856	790	732	679	628	582	543	507	0.214
	6	1638	1489	1359	1246	1149	1063	916	847	786	731	682	637	594	556	0.190
	7	1704	1557	1427	1313	1213	1125	973	902	839	782	731	684	642	603	0.171
	8	1764	1619	1490	1375	1274	1185	1029	955	890	831	777	729	685	645	0.155
	9	1818	1677	1550	1435	1333	1242	1082	1007	939	878	823	773	727	685	0.142
	10	1868	1731	1605	1491	1388	1296	1133	1056	986	924	867	815	768	725	0.131
36/4 Dn = 813	0	841	741	659	511	445	390	344	305	271	241	215	191	171	153	0.728
	1	947	840	753	608	538	475	422	-	-	-	-	-	-	-	0.509
	2	1043	932	839	689	615	558	499	448	404	365	331	301	275	-	0.391
	3	1128	1016	920	764	690	625	569	519	470	427	389	355	326	300	0.318
	4	1204	1092	995	834	757	689	630	577	530	488	447	410	378	349	0.267
	5	1272	1161	1064	898	819	749	687	631	582	537	497	460	426	395	0.231
	6	1331	1224	1127	958	877	805	740	683	631	585	542	503	468	434	0.203
	7	1384	1280	1185	1012	930	857	791	732	678	630	585	545	507	472	0.181
	8	1431	1331	1238	1062	979	905	838	778	722	672	626	584	545	508	0.164
	9	1473	1377	1286	1107	1025	950	882	820	764	712	665	621	580	542	0.149
	10	1509	1418	1330	1148	1067	992	923	861	803	750	701	656	614	574	0.137
	I (in4/ft)	Nominal Shear Due to Panel Buckling, Sn (plf) / Deck Span (ft - in)														
		3 - 0	3 - 6	4 - 0	4 - 6	5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	7 - 6	8 - 0	8 - 6	9 - 0	9 - 6	
	0.1754	15685	11523	8823	6971	5646	4667	3921	3341	2881	2510	2206	1954	1743	1564	

538 PLF / 2.15 = 250 PLF
(ALLOWABLE SHEAR)

Notes:

1. Diaphragm shear and stiffness values are based on the Steel Deck Institute Diaphragm Design Manual, Fourth Edition (DDM04) and AISI S310-2016 (S310-16).
2. Diaphragm shear and stiffness values are based on minimum 3 span condition.
3. An asterisk (*) denotes span may be limited by shear buckling. See bottom table.
4. Shear strength values shown in RED do not comply with minimum SDI side lap connection requirements and shall not be used except with properly spaced side lap connections.

Diaphragm Stiffness, G' (k/in.)

K_2 = 870 kip/in.
K_4 = 3.518
L_v = Span (ft)

$$G' = \frac{K_2}{K_4 + \frac{0.30Dn}{L_v} + 3K_1L_v}$$

Chavez-Grieves
consulting engineers, llc.
A Salas O'Brien Company
4700 Lincoln Road NE, Albuquerque, NM 87109
505.344.4080

SHEET NO.	_____ OF _____		
PROJECT	PROJECT SUNSHINE		
SUBJECT	PUMP HOUSE - CMU WALL DESIGN		
CLIENT	CLIENT	JOB NO.	S99-105
CALCULATED BY	LM	DATE	08/13/2026
CHECKED BY	EOR	DATE	XX/XX/XX

CMU WALL LOADING

CMU WALL DESIGN - SEE RAM ELEMENT CMU WALL MODULE

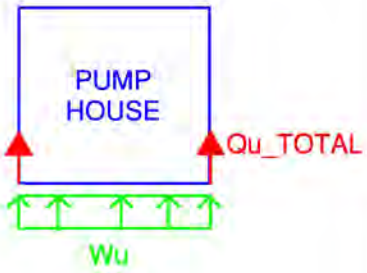
GRAVITY LOADS:
VERTICAL REACTIONS FROM *RAM TRUSS DESIGN REPORT*
DEAD LOAD = 1.0 KIP
LIVE LOAD = 1.24 KIP

WALL WIND PRESSURE:
MWFRS = 40 PSF (WINDWARD AND LEEWARD)
C&C = 39 PSF (OUT OF PLANE WIND PRESSURE)

WALL LATERAL SHEAR REACTION:
Wu_WIND_WALL = 1/2 * 10 FT (HEIGHT) * 40 PSF = 0.20 KLF
Qu_WIND_WALL = 1/2 * 0.20 KLF * 28 FT = 2.8 KIP

ROOF SHEAR REACTION
Qu_WIND_ROOF = 1/2 * 2.83 KIP + 2.62 KIP = 2.7 KIP
ROOF WIND LOADS FROM VALUES FROM *TEKLA TEDDS* MWFRS REPORT

Qu_TOTAL = 2.8 + 2.7 KIP = 5.5 KIP



SEISMIC LOAD: 10% OF TOTAL LOAD
8" CMU WALL DL = 28 (LENGTH) * 10 FT (WEIGHT) * 4 (WALLS) * 47 PSF = 53 KIP
ROOF DL = 32 FT * 32 FT * 20 PSF = 21 KIP
EQ_TOTAL = 1/2 * (53 KIP + 21 KIP) * 10% = 3.7 KIP

DIAPHRAGM DESIGN LOADS:
ASD LOADS

SHEAR = 0.6 * 5.5 KIP / 28 FT = 120 PLF

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505.344.4080 505-343-8759 (fax)

SHEET NO.		OF	
PROJECT	PROJECT NAME		
SUBJECT	SUBJECT		
CLIENT	CLIENT	JOB NO.	NUMBER
CALCULATED BY	ENGR	DATE	XX/XX/XX
CHECKED BY	EOR	DATE	XX/XX/XX

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SHEET NO.		OF	
PROJECT	AUBRUNDALE		
SUBJECT	PUMP HOUSE - CFMF TRUSS LOADS		
CLIENT		JOB NO.	S99-105
CALCULATED BY	ENGR	DATE	XX/XX/XX
CHECKED BY	EOR	DATE	XX/XX/XX

CFMF TRUSS LOADS

GRAVITY LOADS:
- TOP CHORD DEAD LOAD = 12 PSF
- TOP CHORD LIVE LOAD = 20 PSF
- BOTTOM CHORD DEAD LOAD = 5 PSF
- BOTTOM CHORD LIVE LOAD = 5 PSF

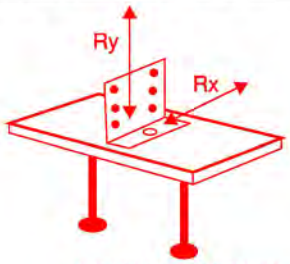
C&C WIND LOADS
- EAVE WIND = -79 PSF, 31.9 PSF
- TOP CHORD = -35.5 PSF,+17.4 PSF
- X/Z PRSSURE ON TOP CHORD = +/- 42.8 PSF
- REFER TO ATTACHED TEDDS OUTPUT FOR C&C WIND LOADS

SEISMIC LOADS:
- 10% MASS LOAD IN EACH X AND Z DIRECTION

MAX ASD REACTION OF BOTTOM CHORD:
- Rx,Rz=2.88K
- Ry=2.6K, -0.6K

SEE ATTACHED MOMENT CLIP CONNECTION CRITERIA AND EMBED PLATE DESIGN CALCULATIONS.

TYPICAL TRUSS CONNECTION TO ALSO BE DESIGNED FOR THESE LOADS.





ASSUMING THESE LOADS ACT SIMULTANEOUSLY THE SCREW CONNECTION SHOULD BE DESIGNED FOR 3.88K

FOR THE GOVERNING TOP CHORD TO BOTTOM CHORD CONNECTION THE ALLOWABLE DESIGN LOAD IS 3.88K IN A SHEAR/ BEARING APPLICATION. THE SHEAR CAPACITY CONTROLS THE SCREW DESIGN

3.88K/867# = 5 SCREWS
6 SCREWS MEANS 5.2K CAPACITY.

IF WE ASSUME 2.88K GOES LATERALLY INTO THE CONNECTION THEN TENSION CAPACITY WOULD CONTROL.

2.88K = 3K / 450# = 7 SCREWS, USE 8 TOTAL SCREWS FOR CONNECTION.

WEB MEMBERS OF TRUSS ARE 43 MILL

MAIN CHORD MEMBERS AND BLOCKING ARE 97 MILL

AISI CALCULATED ALLOWABLE LOADS FOR SCREW CONNECTION

Material thickness (mils)	Design thickness (in)	Material Strength		#8-18 HWH Screw Dia. = 0.160		#10-16 HWH Screw Dia. = 0.190		#12-14 HWH Screw Dia. = 0.210		1/4"-14 HWH Screw Dia. = 0.240	
		Fy (ksi)	Fu (ksi)	Shear (lbs)	Tension (lbs)	Shear (lbs)	Tension (lbs)	Shear (lbs)	Tension (lbs)	Shear (lbs)	Tension (lbs)
33	0.0346	33	45	162	71	177	84	186	93	199	106
43	0.0451	33	45	241	92	263	109	277	121	296	138
54	0.0566	33	45	333	115	370	137	389	152	416	173
	0.0566	50	65	333	167	467	198	562	219	600	250
68	0.0713	33	45	—	—	467	173	550	191	588	218
	0.0713	50	65	—	—	467	249	667	276	849	315
97	0.1017	33	45	—	—	467	246	667	272	867	311
	0.1017	50	65	—	—	467	356	667	393	867	450
118	0.1242	33	45	—	—	—	—	667	333	867	380
	0.1242	50	65	—	—	—	—	667	480	867	549

AISI CALCULATED ALLOWABLE BEARING & PULLOVER FOR SCREWS

Material thickness (mils)	Design thickness (in)	Material Strength		#8-18 Screw Shank = 0.160 Head = 0.250		#10-16 Screw Shank = 0.190 Head = 0.375		#12-14 Screw Shank = 0.210 Head = 0.375		1/4"-14 Screw Shank = 0.240 Head = 0.500	
		Fy (ksi)	Fu (ksi)	Bearing (lbs)	Pullover (lbs)	Bearing (lbs)	Pullover (lbs)	Bearing (lbs)	Pullover (lbs)	Bearing (lbs)	Pullover (lbs)
33	0.0346	33	45	224	195	266	292	294	292	336	389
43	0.0451	33	45	292	254	347	381	384	381	438	507
54	0.0566	33	45	367	318	436	478	481	478	550	637
		50	65	530	460	629	690	695	690	795	920
68	0.0713	33	45	—	—	549	602	606	602	693	802
		50	65	—	—	792	869	876	869	1001	1159
97	0.1017	33	45	—	—	783	858	865	858	989	1144
		50	65	—	—	1130	1239	1249	1239	1428	1653
118	0.1242	33	45	—	—	—	—	1056	1048	1207	1397
		50	65	—	—	—	—	1526	1514	1744	2018

Notes:
1 All values were calculated using the 2001 AISI Specification w/2004 supplement.
2 Charts are based on Buildex TEK2 HWH screw capacities. All screws must meet minimum criteria outlined.
3 Shear strength for #8, #10, #12, and 1/4" screws must be greater than or equal to 1000 lbs, 1400 lbs, 2000 lbs and 2600 lbs respe
4 Tension strength for #8, #10, #12, and 1/4" screws must be greater than or equal to 1545 lbs, 1936 lbs, 2778 lbs and 4060 lbs respe
5 The minimum head diameter for #8 screws is 1/4". The minimum head diameter for #10 and #12 screws is 3/8". The minimum head di
6 Screw ultimate shear capacity is based on Buildex® DATA as a minimum.
7 Buildex is a registered trademark of Illinois Tool Works, Inc.

TYPICAL WEB MEMBER TO CHORD HAS A MAX REACTION OF :
- ANGLED WEB AXIAL LOAD=2.21K
- VERT WEB AXIAL LOAD =1.25K

USE A MIN 4-1/4" DIA SCREWS = 2.4K = 92% DCR

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4700 Lincoln Road NE, Albuquerque, NM 87109
505.344.4080 505-343-8759 (fax)

SHEET NO.		OF	
PROJECT	AUBRUNDALE		
SUBJECT	PUMP HOUSE - CFMF TRUSS LOADS		
CLIENT		JOB NO.	S99-105
CALCULATED BY	ENGR	DATE	XX/XX/XX
CHECKED BY	EOR	DATE	XX/XX/XX

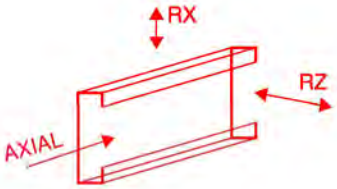
CFMF TRUSS LOADS

CONNECTION DESIGNS:

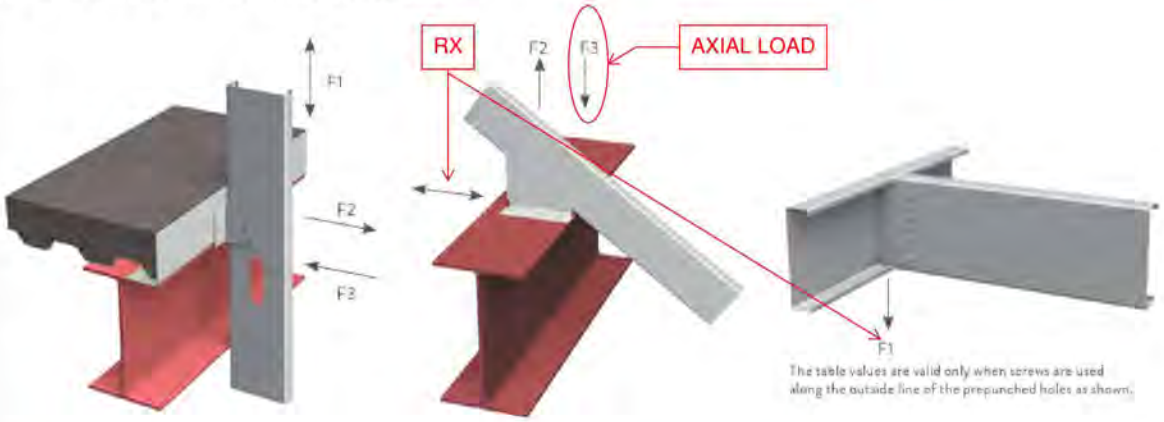
PREVIOUS PAGE DEPICTS MINIMUM SCREWS NEEDED FOR THE CONTROLLING DESIGN FOR BOTH CHORD AND WEB MEMBERS.

TOP AND BOTTOM BRACING MEMBER TO ATTACH TO THE CHORDS VIA A RIGID CLIP, IN SOME INSTANCES THE CLIPS WILL BE SKEWED. SEE CLIP CAPACITY BELOW. NOTE THE CONTROLLING DESIGN FOR THE BRACING MEMBERS IS AXIAL DEMAND DUE TO THE ROOF SHAPE, SHER AND MOMENT VALUES ARE SIGNIFICANTLY LOAD WHEN COMPARED TO THE AXIAL WIND LOAD.

BOTTOM BRACING CONTROLLING MEMBER LOADS:
DCR = 0.04, LC=D2
RX=0.02K (F4)
RZ=0K (F2)
AXIAL=1.34K
TOP BRACING CONTROLLING MEMBER LOADS:
DCR=0.15, LC=D2
RX=0.15K (F4)
RZ=0K (F2)
AXIAL= 2.25K



AXIAL LOAD IS RESOLVED BY COMPRESSION OF THE MEMBER AND CONNECTION TRANSFER.



The table values are valid only when screws are used along the outside line of the prepunched holes as shown.

E-Series™ Support Clips Allowable Clip Capacities (lbs)

USING #10-16 SELF-DRILLING SCREWS

Clip Designation	No. of Screws to Steel Framing	Stud Thickness and Yield Strength: 20ga (33mils) 33ksi F1	20ga (33mils) 33ksi F2	20ga (33mils) 33ksi F3	18ga (43mils) 33ksi F1	18ga (43mils) 33ksi F2	18ga (43mils) 33ksi F3	16ga (54mils) 50ksi F1	16ga (54mils) 50ksi F2	16ga (54mils) 50ksi F3
ES43	3	101 (101)	210 (531)	507	150 (150)	210 (788)	507	266 (155)	210 (1195)	507
ES45	2	176 (176)	354 (354)	354	261 (261)	371 (525)	525	463 (453)	371 (933)	811
	5	251 (251)	371 (885)	885	372 (372)	371 (1313)	912	629 (479)	371 (2105)	912
ES47	4	380 (380)	531 (708)	708	564 (564)	531 (1050)	1050	1002 (970)	531 (1867)	1347
	7	455 (455)	531 (1239)	1239	675 (675)	531 (1838)	1318	1169 (960)	531 (3015)	1318
ES49	4	477 (477)	692 (708)	708	707 (707)	692 (1050)	1050	1257 (1257)	692 (1867)	1753
	9	706 (706)	692 (1593)	1593	1048 (1048)	692 (2363)	1724	1862 (1576)	692 (3925)	1724
ES41	6	727 (727)	852 (1062)	1062	1079 (1079)	852 (1576)	1576	1918 (1918)	852 (2800)	2053
	11	995 (995)	852 (1947)	1947	1476 (1476)	852 (2889)	2130	2623 (2301)	852 (4835)	2130
E683	3	101 (101)	333 (531)	531	150 (150)	333 (788)	788	266 (196)	333 (1400)	1011
E685	2	176 (176)	354 (354)	354	261 (261)	525 (525)	525	463 (463)	587 (933)	933
	5	251 (251)	587 (885)	885	372 (372)	587 (1313)	1313	661 (602)	587 (2333)	1817
E687	4	380 (380)	708 (708)	708	564 (564)	841 (1050)	1050	1002 (1002)	841 (1867)	1867
	7	455 (455)	841 (1239)	1239	675 (675)	841 (1838)	1838	1200 (1200)	841 (3267)	2625
E689	4	477 (477)	708 (708)	708	707 (707)	1050 (1050)	1050	1257 (1257)	1095 (1867)	1867
	9	706 (706)	1095 (1593)	1593	1048 (1048)	1095 (2363)	2363	1862 (1862)	1095 (4200)	3434
E681	6	727 (727)	1062 (1062)	1062	1079 (1079)	1349 (1576)	1576	1918 (1918)	1349 (2800)	2800
	11	995 (995)	1349 (1947)	1947	1476 (1476)	1349 (2889)	2889	2623 (2623)	1349 (5133)	4244
E973	3	101 (101)	531 (531)	531	150 (150)	679 (788)	788	266 (266)	679 (1400)	1400
E975	2	176 (176)	354 (354)	354	261 (261)	525 (525)	525	463 (463)	933 (933)	933
	5	251 (251)	885 (885)	885	372 (372)	1196 (1313)	1313	661 (661)	1196 (2333)	2333
E977	4	380 (380)	708 (708)	708	564 (564)	1050 (1050)	1050	1002 (1002)	1713 (1867)	1867
	7	455 (455)	1239 (1239)	1239	675 (675)	1713 (1838)	1838	1200 (1200)	1713 (3267)	3267
E979	4	477 (477)	708 (708)	708	707 (707)	1050 (1050)	1050	1257 (1257)	1867 (1867)	1867
	9	706 (706)	1593 (1593)	1593	1048 (1048)	2229 (2363)	2363	1862 (1862)	2229 (4200)	4200
E971	6	727 (727)	1062 (1062)	1062	1079 (1079)	1576 (1576)	1576	1918 (1918)	2746 (2800)	2800
	11	995 (995)	1947 (1947)	1947	1476 (1476)	2746 (2889)	2889	2623 (2623)	2746 (5133)	5133

ENSURING AXIAL DEMAND IS MET THIS IS THE MINIMUM CONNECTOR FOR AX = 2.3K FOR A BRACING MEMBER

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505.344.4080 505-343-8759 (fax)

SHEET NO.		OF	
PROJECT	AUBRUNDALE		
SUBJECT	PUMP HOUSE - CFMF TRUSS LOADS		
CLIENT		JOB NO.	S99-105
CALCULATED BY	ENGR	DATE	XX/XX/XX
CHECKED BY	EOR	DATE	XX/XX/XX

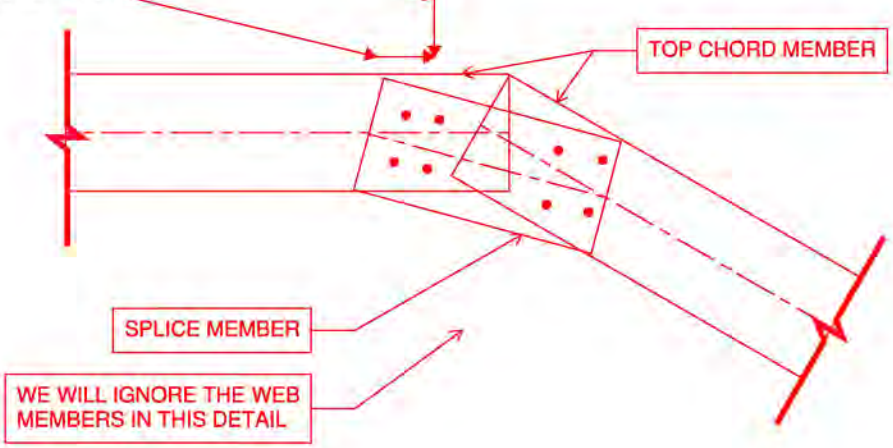
CFMF TRUSS LOADS

CONNECTION DESIGNS:

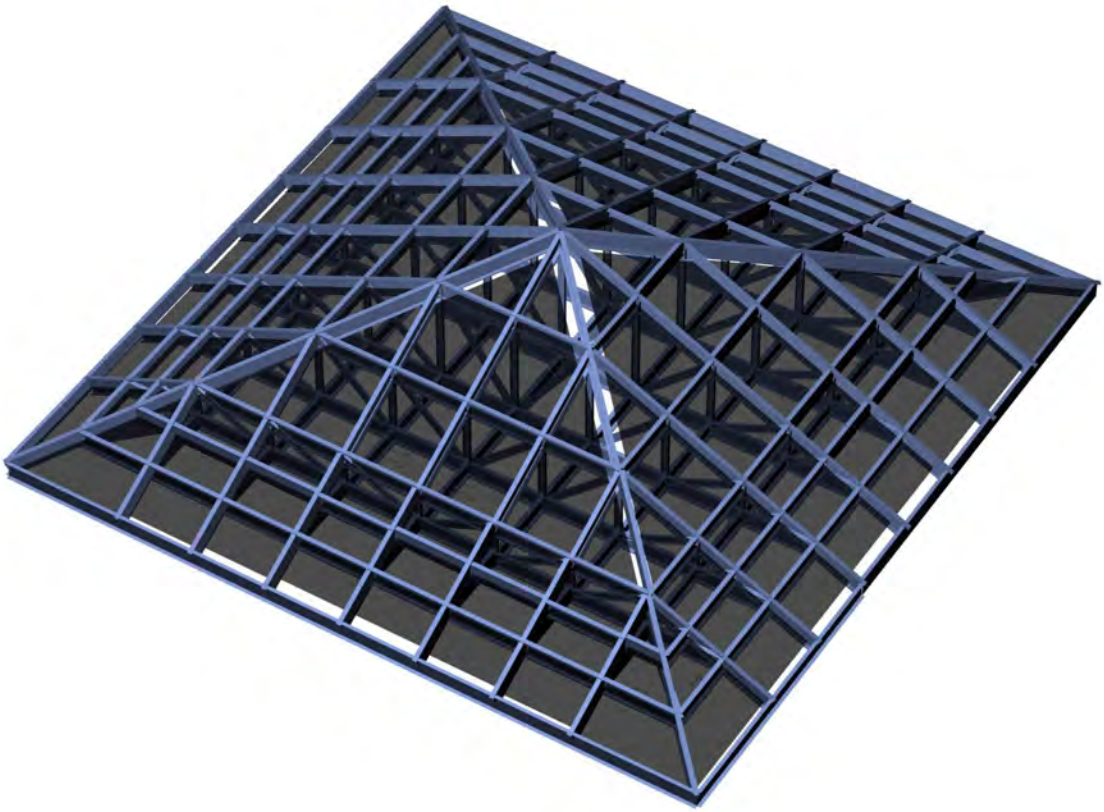
ANGLED TOP CHORD CONNECTION. IT IS NOT WISE TO CUT OR NOTCH THE FLANGED OF THE TOPCHORDS TO MAKE A CONNECTION. TO ENSURE A RIGID CONNECTION IS MADE WE WILL ADD A SPLICE MEMBER FROM FLAT TOP CHORD TO ANGLED TOP CHORD.

TOP CHORD CONTROLLING LOADS:
DCR = 1.04 UNDER ASD LOADING = 0.992 UNDER LRFD LOADING WHICH WAS THE DESIGN BASIS. LC=D8
(DL+0.45WX+0.75LL)
SHEAR IN THE 22=0.25K
SHEAR IN THE 33=0.03K
AXIAL LOAD = 3.2K

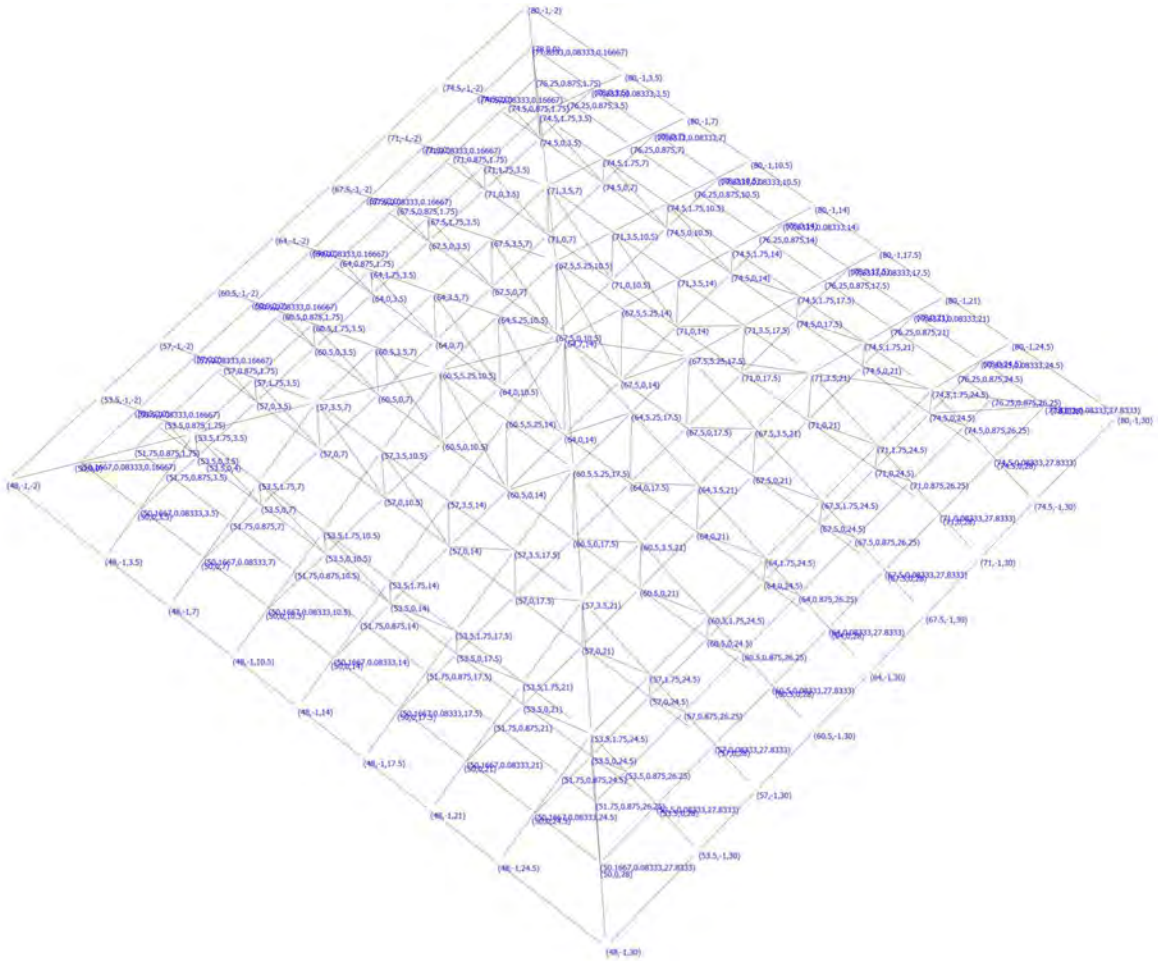
TOP CHORD CONTROLLING LOADS:
DCR = 0.25 UNDER ASD LOADING, LC=D9
(DL+0.45WZ+0.75LL)
SHEAR IN THE 22=0.25K
SHEAR IN THE 33=0.03K
AXIAL LOAD = 1.65K



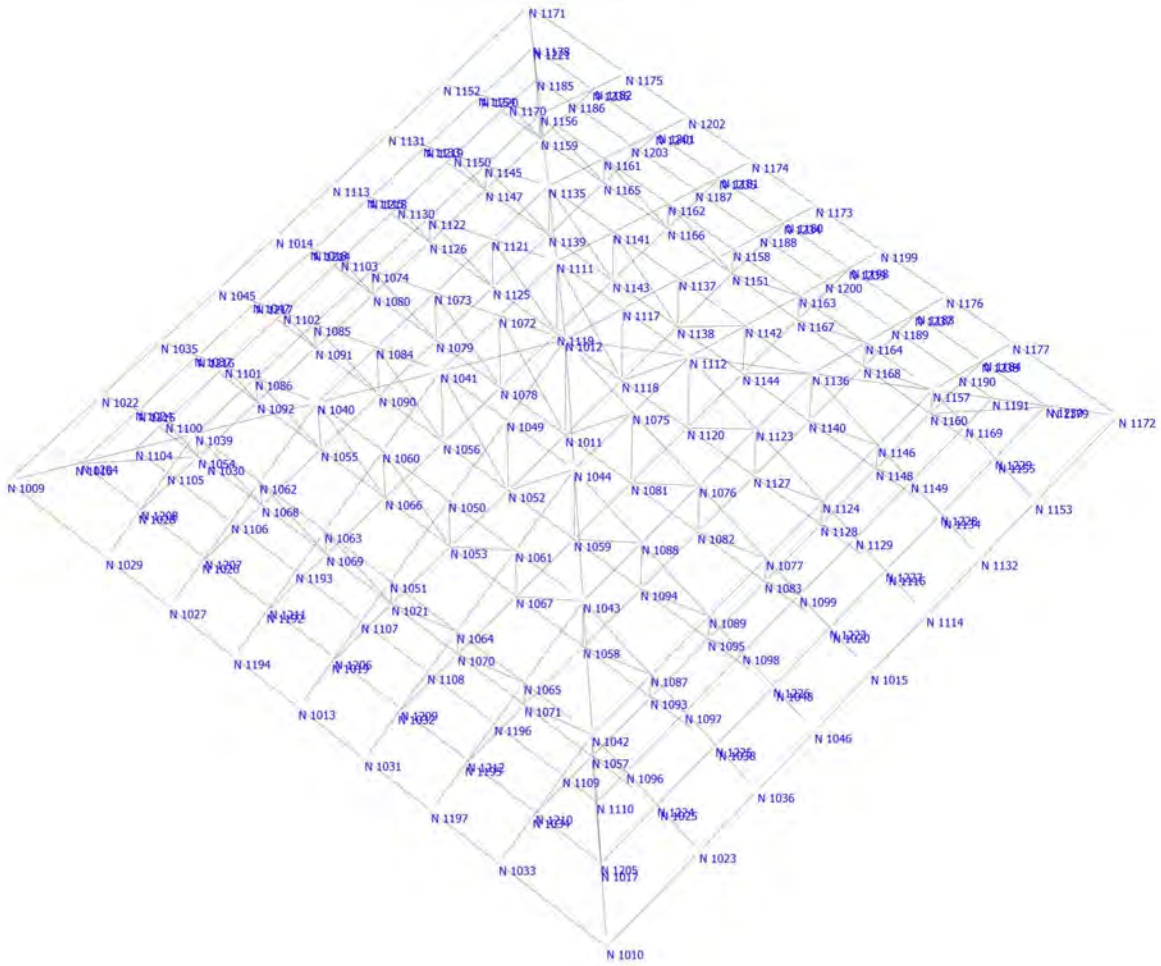
DESIGN SHEAR LOAD IN CONNECTION = 3.3.K
1/4" DIA TEK SCREW SHEAR CAPACITY = 867#/SCREW
3.3K/867#=4 SCREWS, USE 6 AT A MINIMUM


Y
X
Z

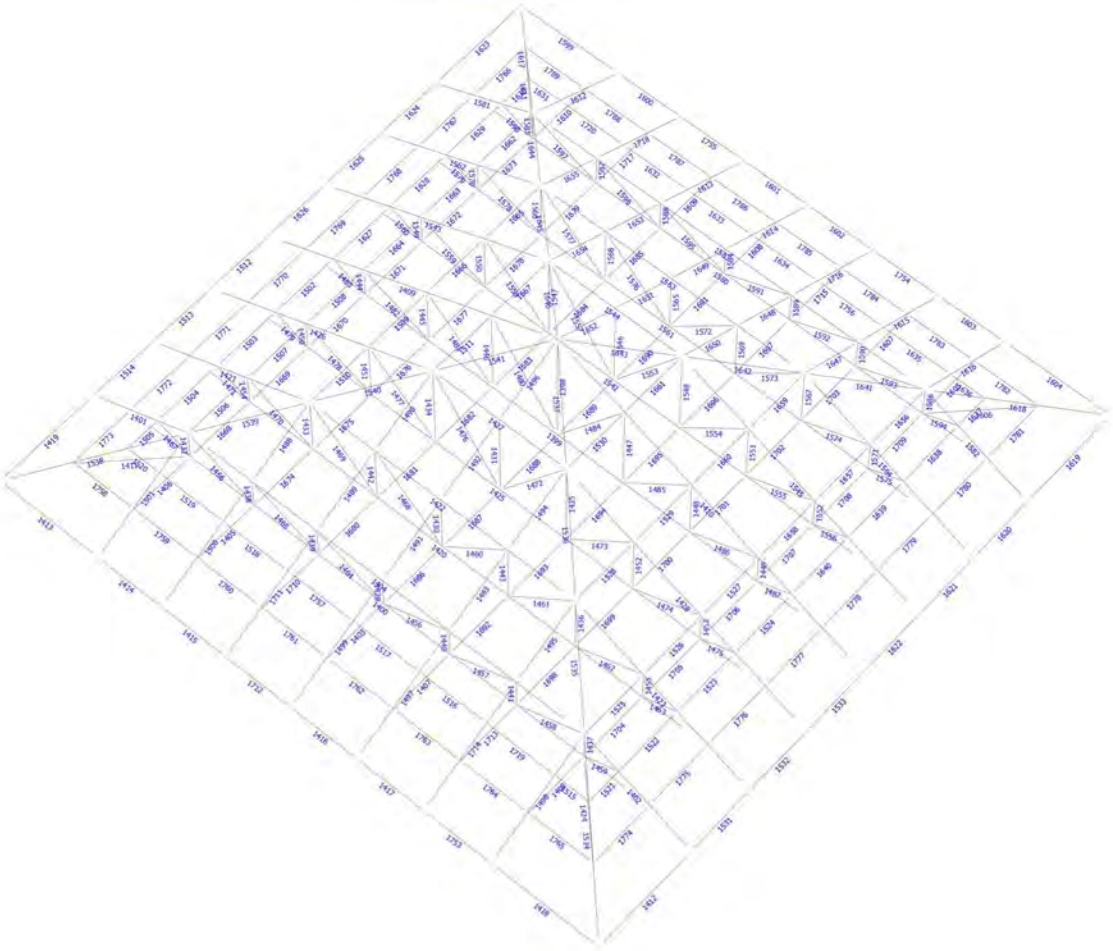
NODAL COORDINATES



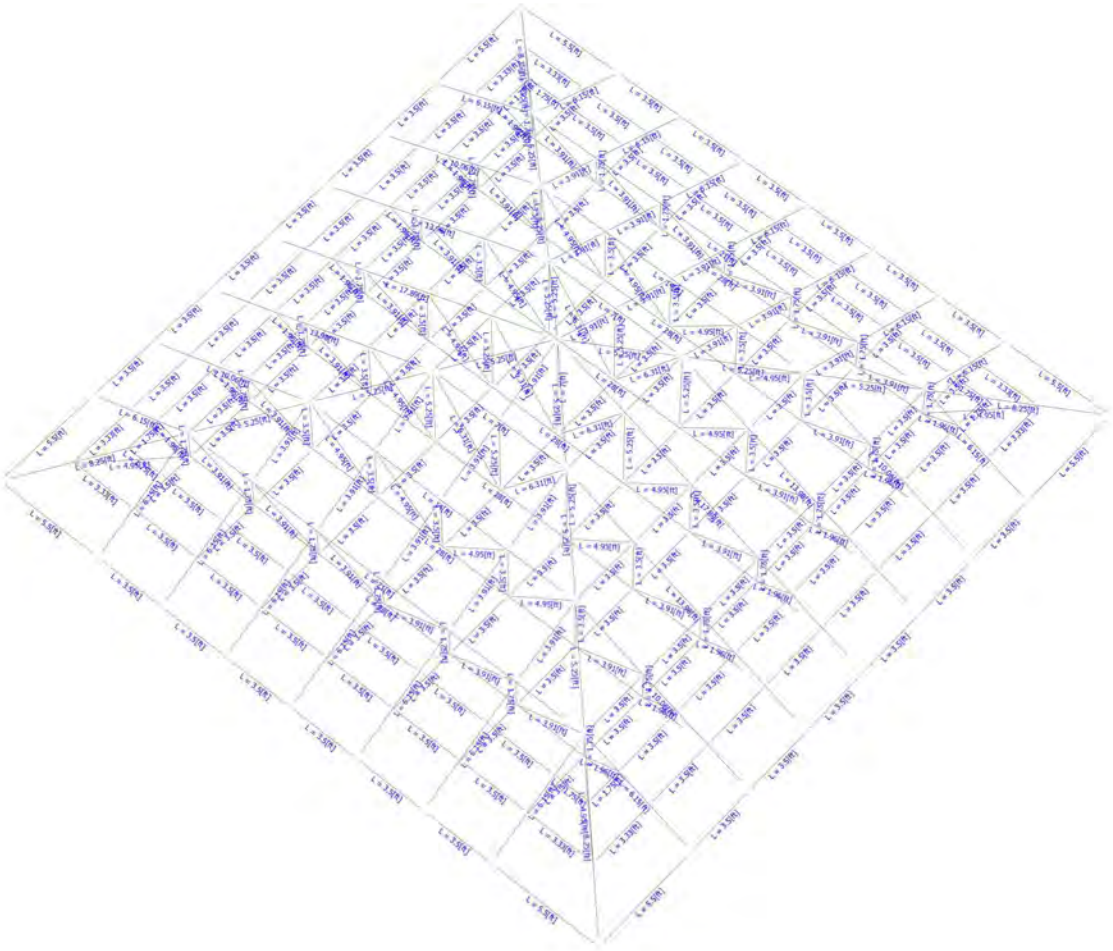
NODAL NUMBERS



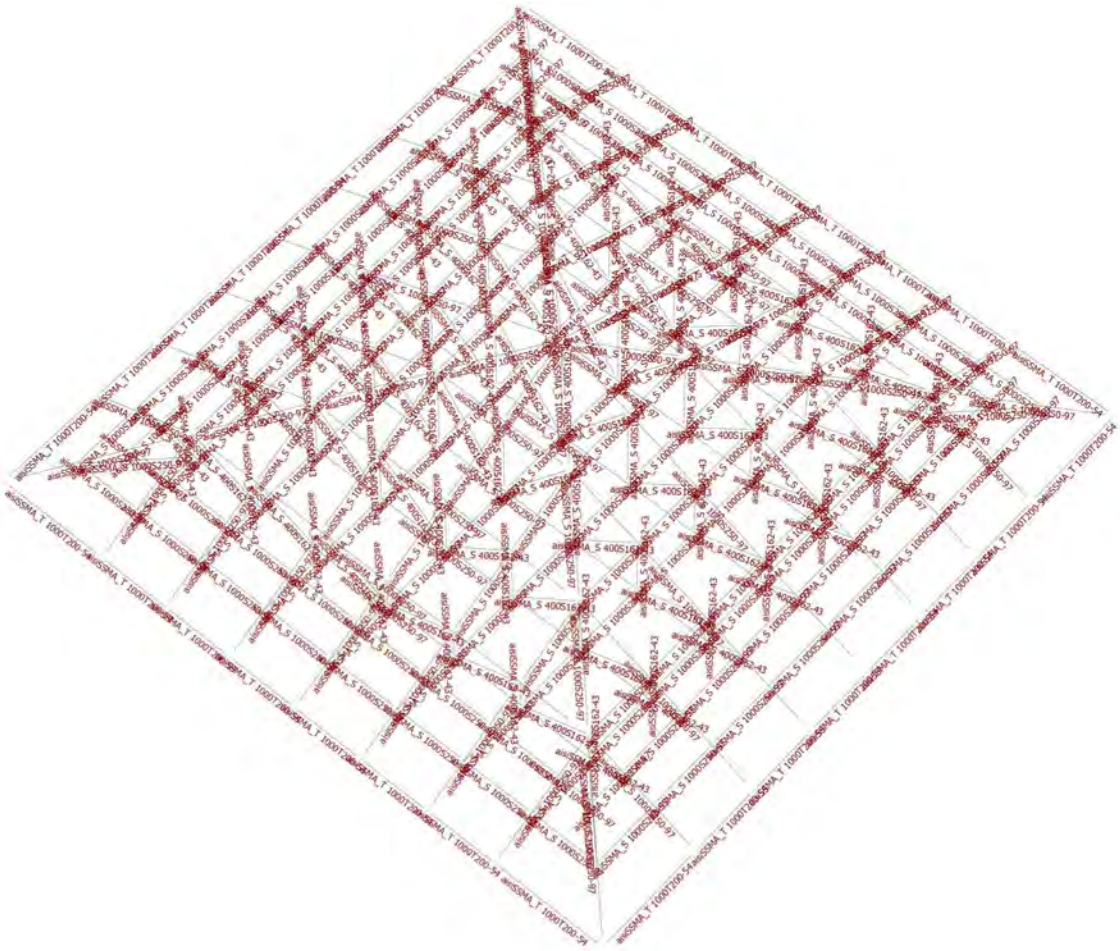
MEMBER NUMBERS



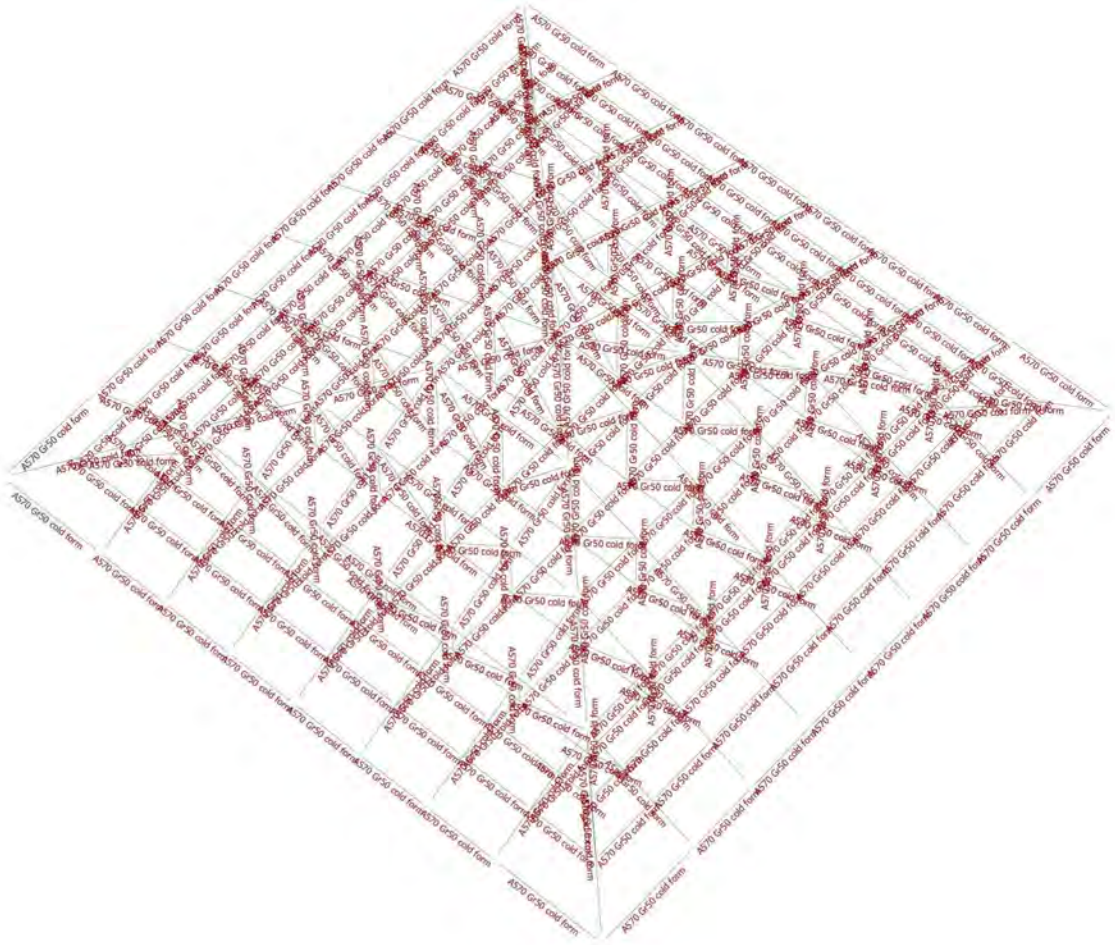
MEMBER LENGTHS



MEMBER SECTIONS

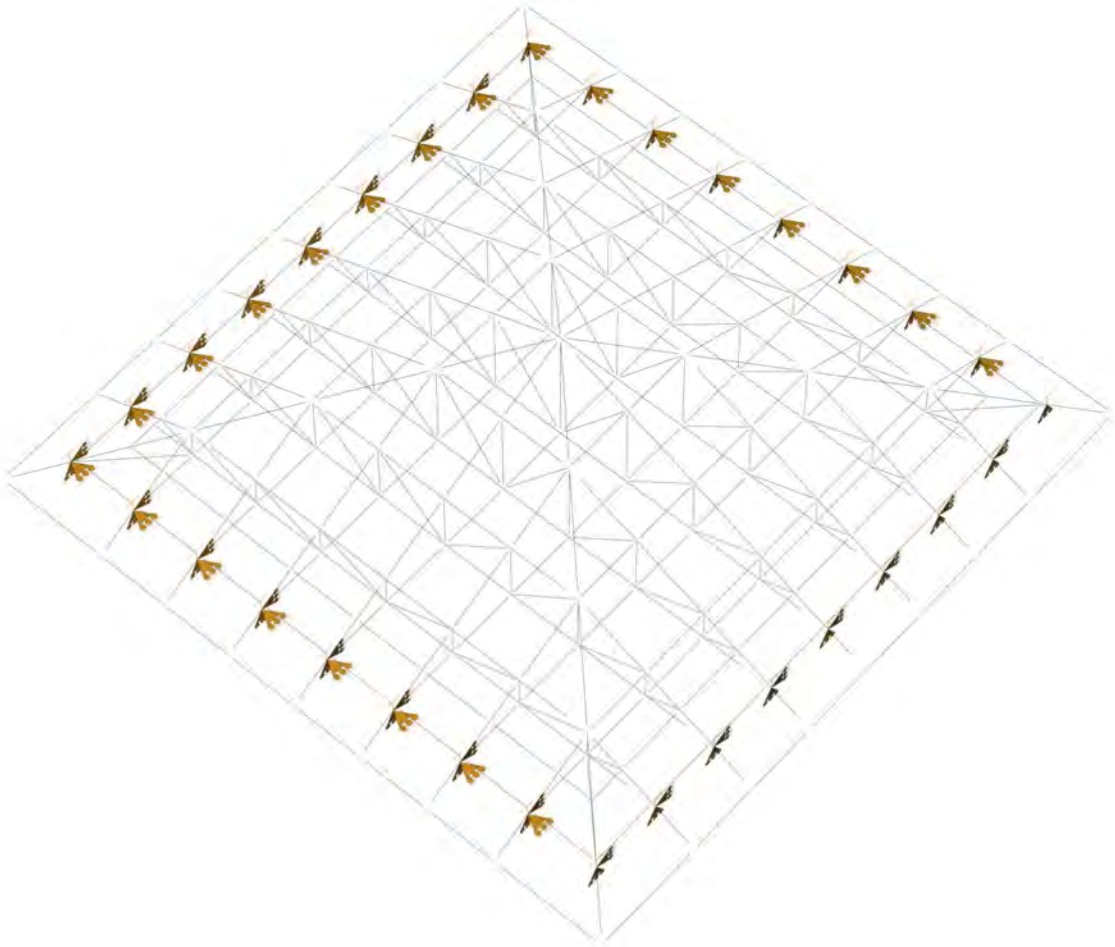


MEMBER PROPERTIES

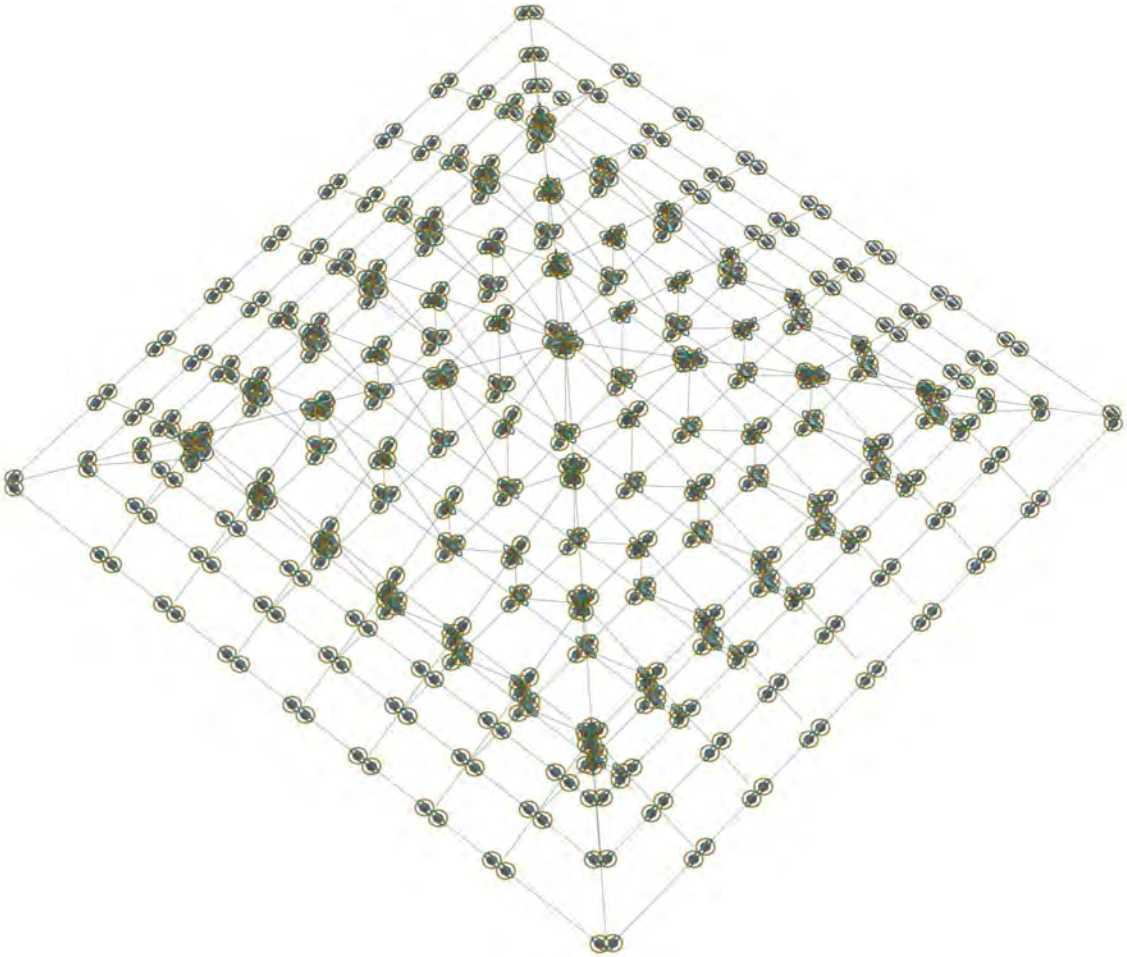




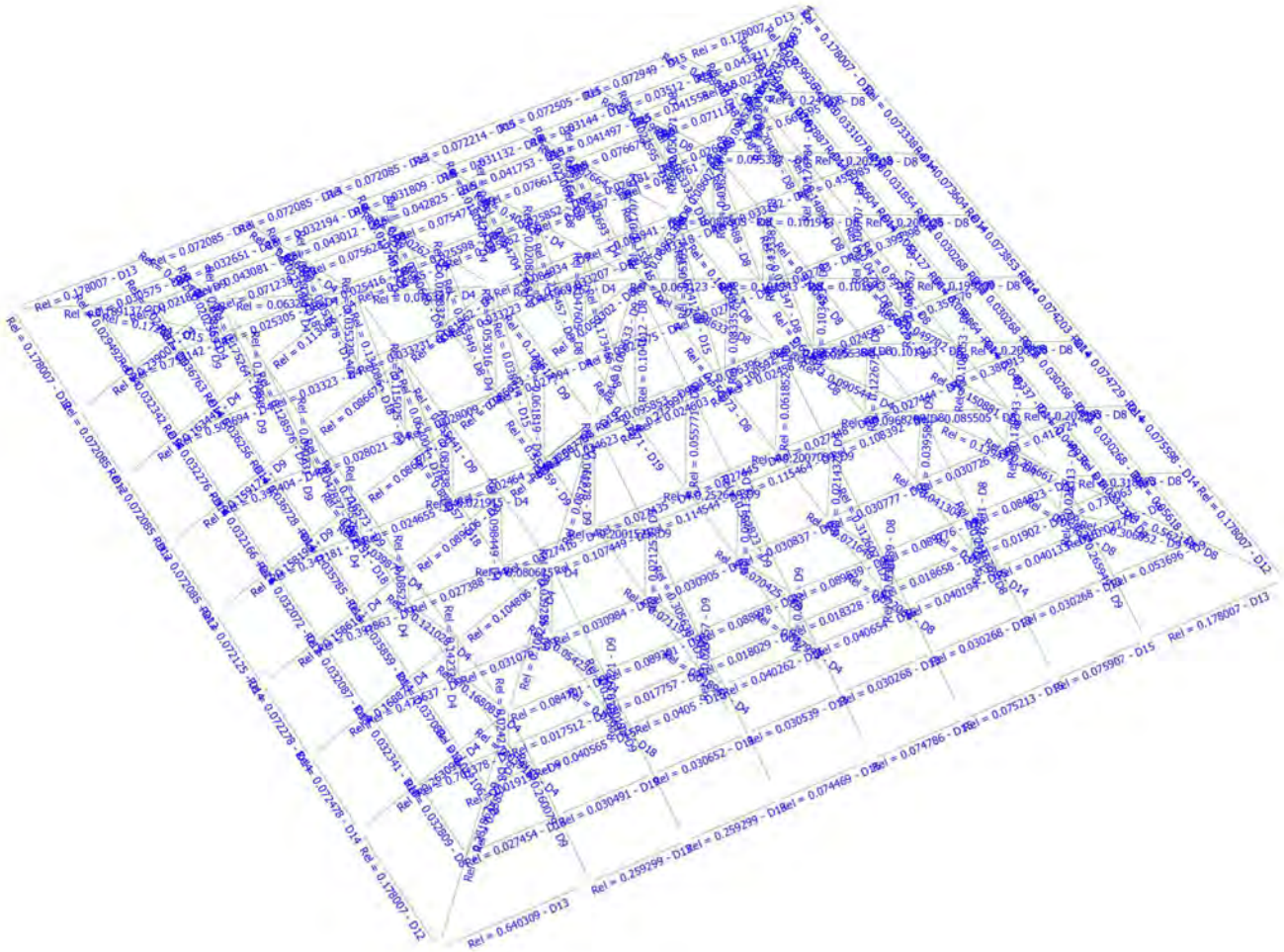
NODAL FIXITIES



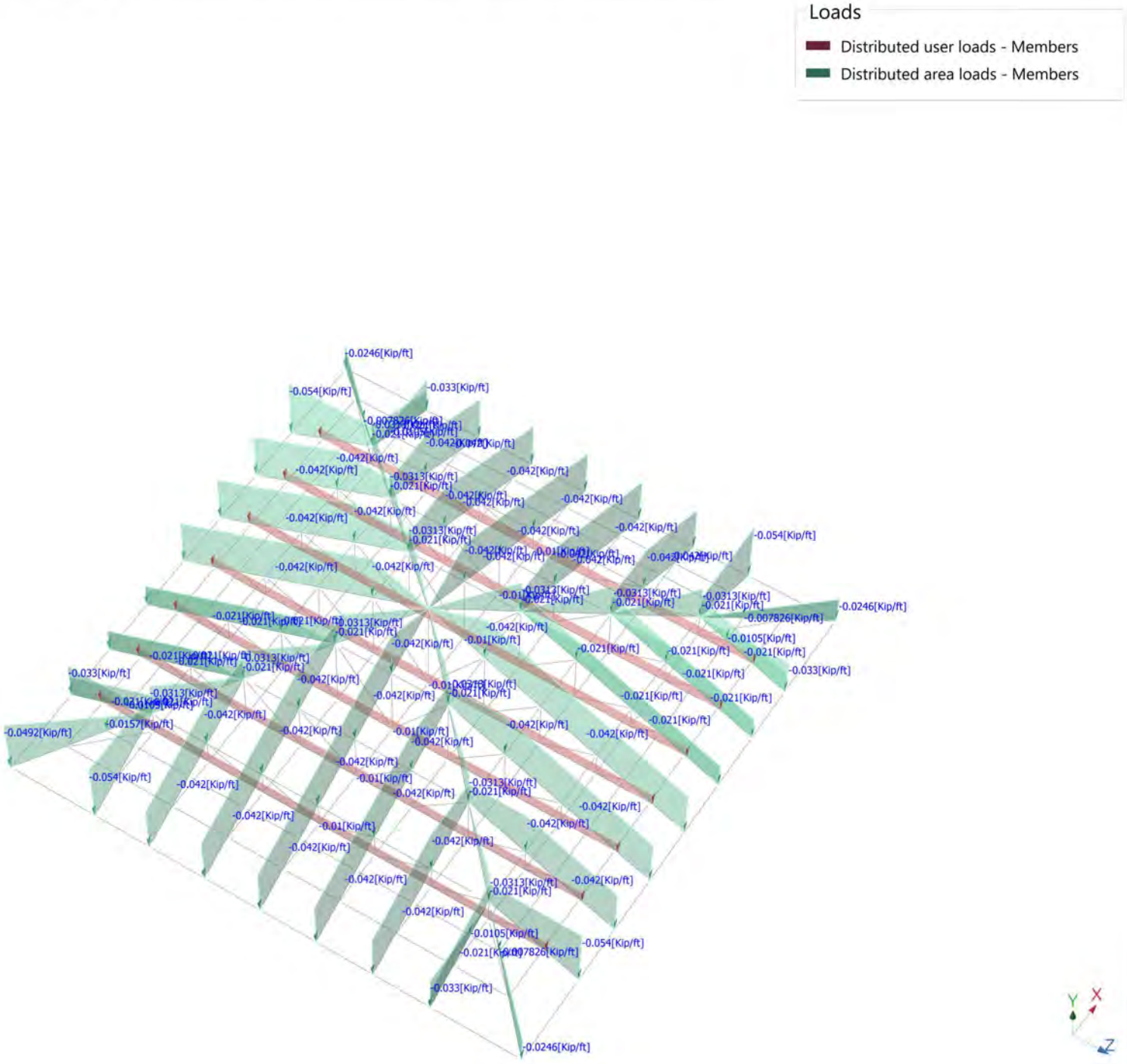
MEMBER RELEASES

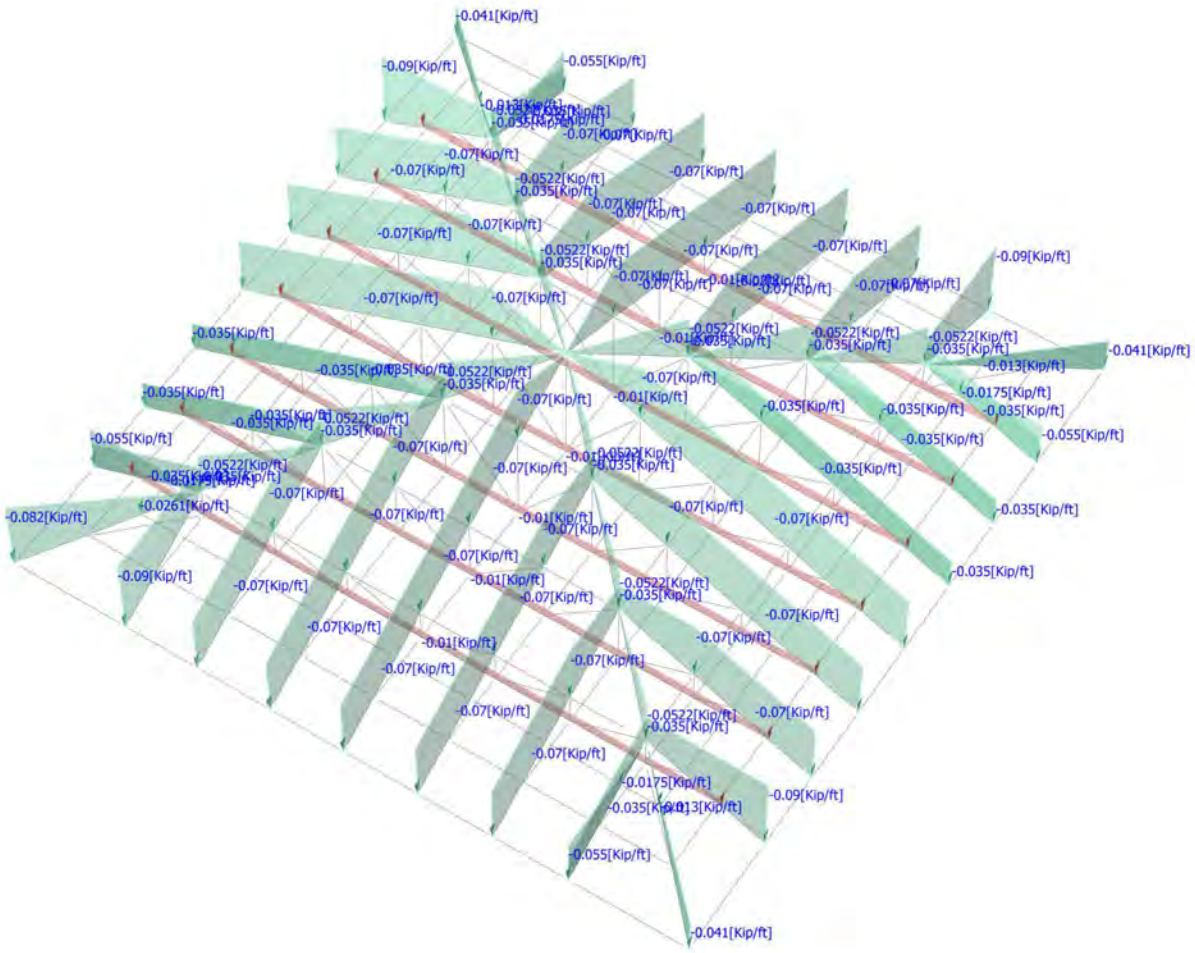


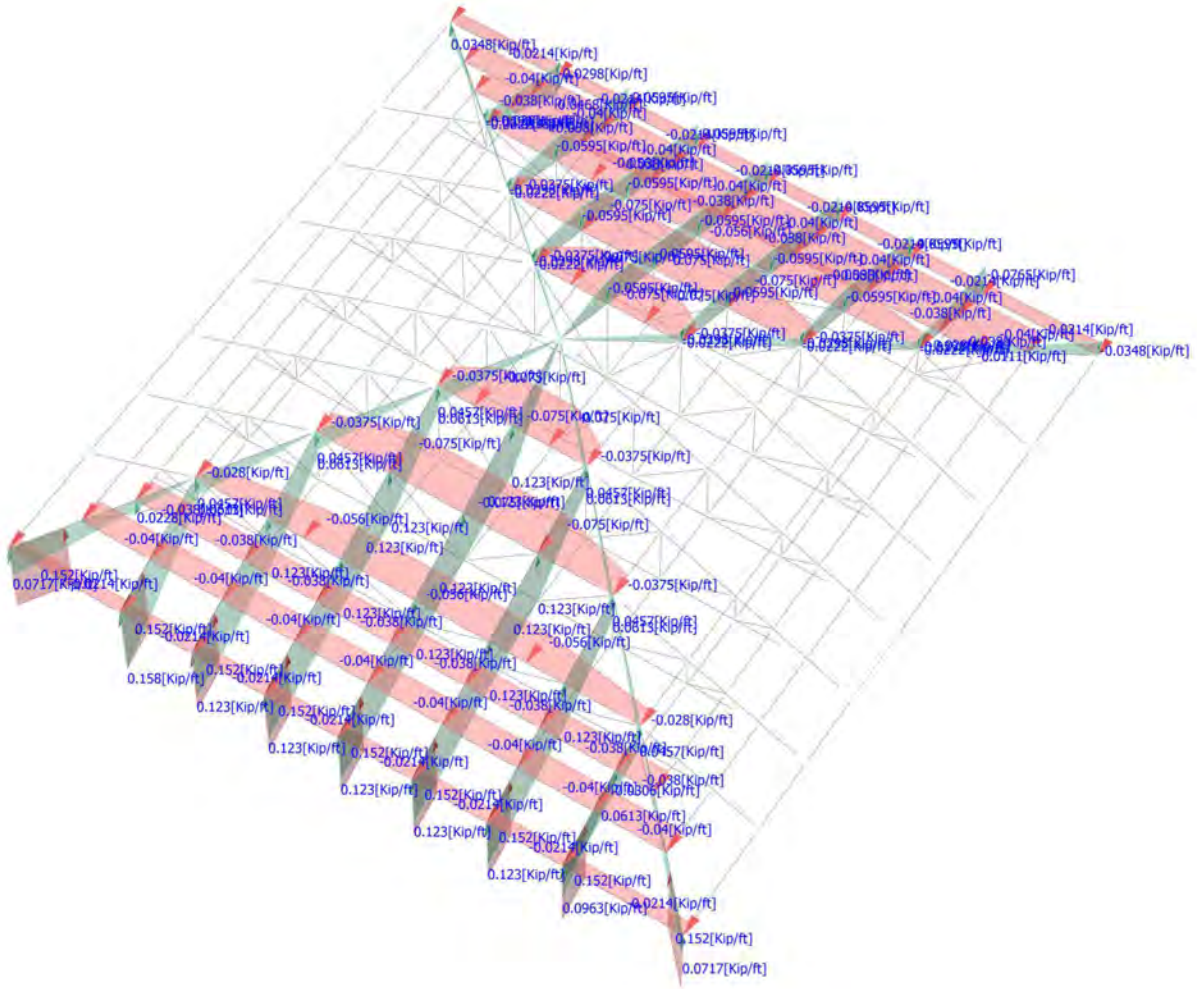
MEMBER STRESS RATIOS



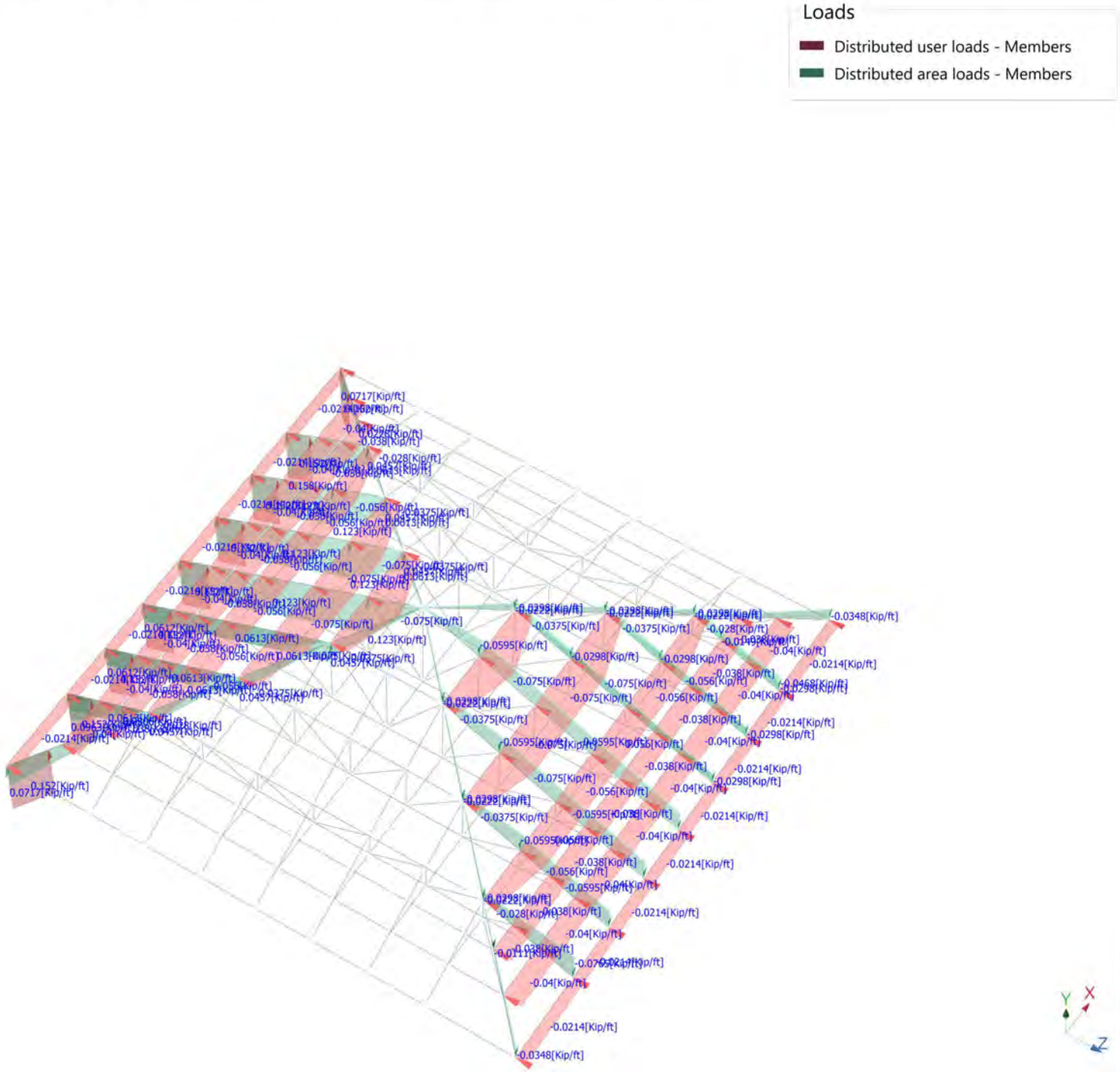
Current Date: 9/15/2026 5:44 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx
Load condition: DL=Dead Load

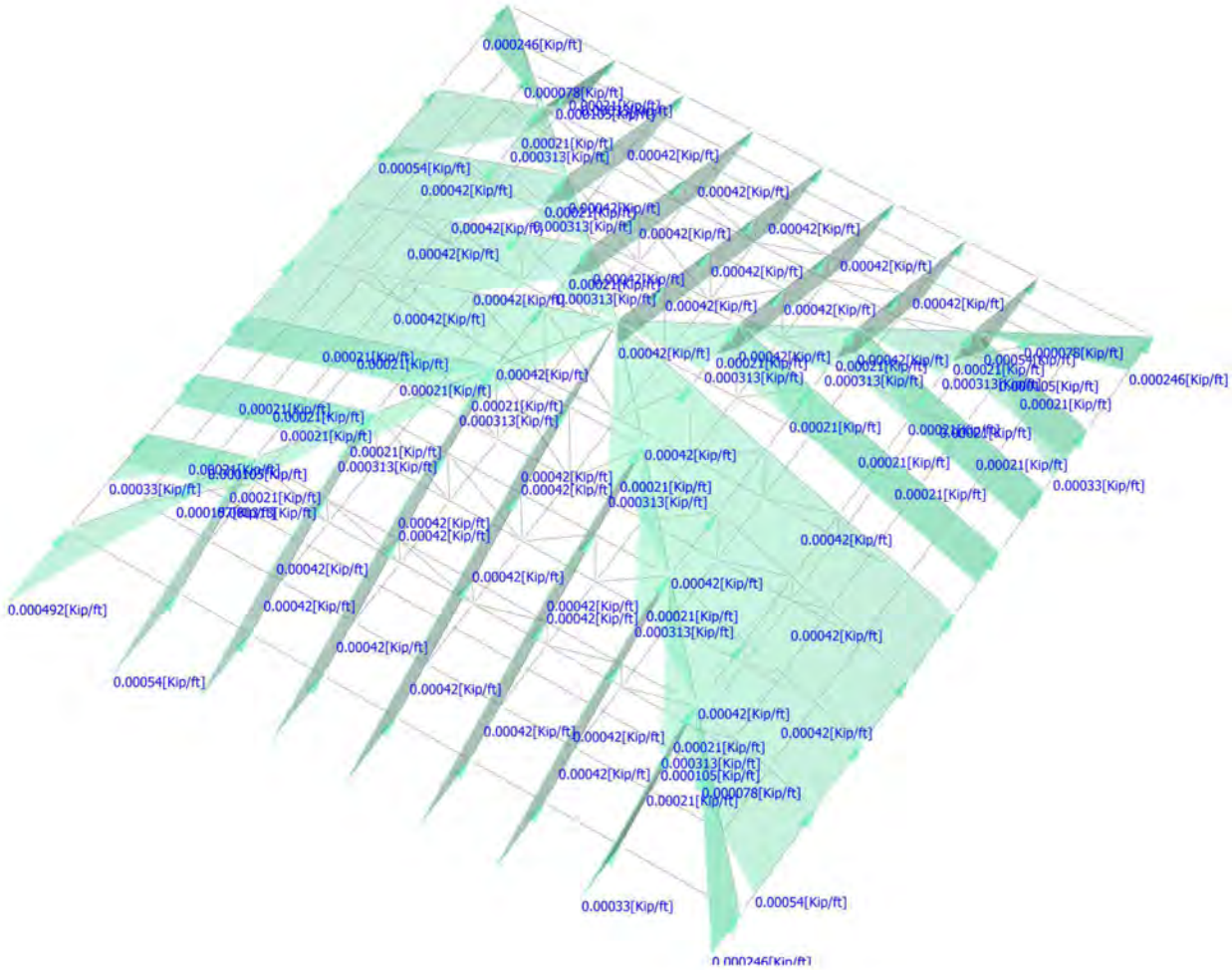


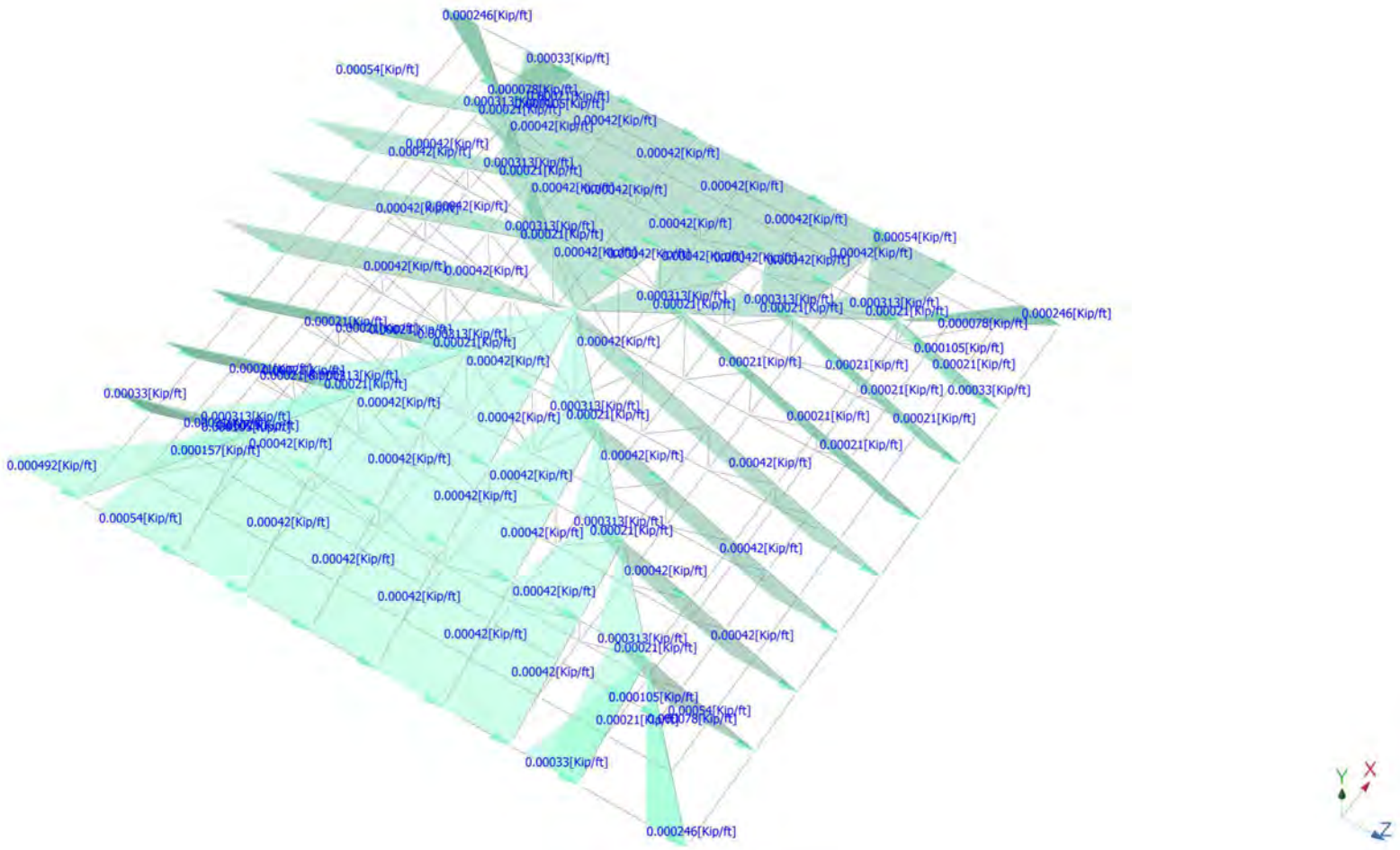

Loads
Distributed user loads - Members
Distributed area loads - Members
-0.041[Kip/ft]
-0.09[Kip/ft]
-0.055[Kip/ft]
-0.07[Kip/ft]
-0.0522[Kip/ft]
-0.035[Kip/ft]
-0.01[Kip/ft]
-0.0175[Kip/ft]
-0.013[Kip/ft]
-0.082[Kip/ft]
+0.0261[Kip/ft]
Y
X
Z


Loads
Distributed user loads - Members
Distributed area loads - Members
0.0348[Kip/ft]
0.0214[Kip/ft]
0.0298[Kip/ft]
-0.04[Kip/ft]
0.038[Kip/ft]
-0.0595[Kip/ft]
0.04[Kip/ft]
-0.0375[Kip/ft]
0.0228[Kip/ft]
-0.075[Kip/ft]
-0.038[Kip/ft]
-0.056[Kip/ft]
-0.058[Kip/ft]
0.0765[Kip/ft]
0.0314[Kip/ft]
0.0111[Kip/ft]
0.0457[Kip/ft]
0.0613[Kip/ft]
0.123[Kip/ft]
-0.028[Kip/ft]
0.0717[Kip/ft]
0.152[Kip/ft]
0.158[Kip/ft]
0.0963[Kip/ft]
0.0306[Kip/ft]
Y
X
Z

Current Date: 9/15/2026 5:45 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx
Load condition: Wz=Wind in Z




Loads
Distributed area loads - Members
0.000246[Kip/ft]
0.000078[Kip/ft]
0.00021[Kip/ft]
0.000313[Kip/ft]
0.00042[Kip/ft]
0.00054[Kip/ft]
0.000105[Kip/ft]
0.00033[Kip/ft]
0.000492[Kip/ft]
Y
X
Z


Loads
Distributed area loads - Members
0.000246[Kip/ft]
0.00033[Kip/ft]
0.00054[Kip/ft]
0.000078[Kip/ft]
0.00042[Kip/ft]
0.000313[Kip/ft]
0.00021[Kip/ft]
0.000105[Kip/ft]
0.000492[Kip/ft]
0.000157[Kip/ft]
Y
X
Z

Current Date: 9/15/2026 5:46 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx

Geometry data

Nodes

Node	X [ft]	Y [ft]	Z [ft]	Rigid Floor
1009	48.00	-1.00	-2.00	0
1010	48.00	-1.00	30.00	0
1011	64.00	0.00	14.00	0
1012	64.00	7.00	14.00	0
1013	48.00	-1.00	14.00	0
1014	64.00	-1.00	-2.00	0
1015	64.00	-1.00	30.00	0
1016	50.00	0.00	0.00	0
1017	50.00	0.00	28.00	0
1018	64.00	0.00	0.00	0
1019	50.00	0.00	14.00	0
1020	64.00	0.00	28.00	0
1021	53.50	0.00	14.00	0
1022	53.50	-1.00	-2.00	0
1023	53.50	-1.00	30.00	0
1024	53.50	0.00	0.00	0
1025	53.50	0.00	28.00	0
1026	50.00	0.00	7.00	0
1027	48.00	-1.00	7.00	0
1028	50.00	0.00	3.50	0
1029	48.00	-1.00	3.50	0
1030	53.50	0.00	4.00	0
1031	48.00	-1.00	17.50	0
1032	50.00	0.00	17.50	0
1033	48.00	-1.00	24.50	0
1034	50.00	0.00	24.50	0
1035	57.00	-1.00	-2.00	0
1036	57.00	-1.00	30.00	0
1037	57.00	0.00	0.00	0
1038	57.00	0.00	28.00	0
1039	53.50	1.75	3.50	0
1040	57.00	3.50	7.00	0
1041	60.50	5.25	10.50	0
1042	53.50	1.75	24.50	0
1043	57.00	3.50	21.00	0
1044	60.50	5.25	17.50	0
1045	60.50	-1.00	-2.00	0
1046	60.50	-1.00	30.00	0
1047	60.50	0.00	0.00	0
1048	60.50	0.00	28.00	0
1049	60.50	5.25	14.00	0
1050	57.00	3.50	14.00	0
1051	53.50	1.75	14.00	0
1052	60.50	0.00	14.00	0
1053	57.00	0.00	14.00	0
1054	53.50	0.00	3.50	0
1055	57.00	0.00	7.00	0
1056	60.50	0.00	10.50	0
1057	53.50	0.00	24.50	0

1058	57.00	0.00	21.00	0
1059	60.50	0.00	17.50	0
1060	57.00	3.50	10.50	0
1061	57.00	3.50	17.50	0
1062	53.50	1.75	7.00	0
1063	53.50	1.75	10.50	0
1064	53.50	1.75	17.50	0
1065	53.50	1.75	21.00	0
1066	57.00	0.00	10.50	0
1067	57.00	0.00	17.50	0
1068	53.50	0.00	7.00	0
1069	53.50	0.00	10.50	0
1070	53.50	0.00	17.50	0
1071	53.50	0.00	21.00	0
1072	64.00	5.25	10.50	0
1073	64.00	3.50	7.00	0
1074	64.00	1.75	3.50	0
1075	64.00	5.25	17.50	0
1076	64.00	3.50	21.00	0
1077	64.00	1.75	24.50	0
1078	64.00	0.00	10.50	0
1079	64.00	0.00	7.00	0
1080	64.00	0.00	3.50	0
1081	64.00	0.00	17.50	0
1082	64.00	0.00	21.00	0
1083	64.00	0.00	24.50	0
1084	60.50	3.50	7.00	0
1085	60.50	1.75	3.50	0
1086	57.00	1.75	3.50	0
1087	57.00	1.75	24.50	0
1088	60.50	3.50	21.00	0
1089	60.50	1.75	24.50	0
1090	60.50	0.00	7.00	0
1091	60.50	0.00	3.50	0
1092	57.00	0.00	3.50	0
1093	57.00	0.00	24.50	0
1094	60.50	0.00	21.00	0
1095	60.50	0.00	24.50	0
1096	53.50	0.875	26.25	0
1097	57.00	0.875	26.25	0
1098	60.50	0.875	26.25	0
1099	64.00	0.875	26.25	0
1100	53.50	0.875	1.75	0
1101	57.00	0.875	1.75	0
1102	60.50	0.875	1.75	0
1103	64.00	0.875	1.75	0
1104	51.75	0.875	1.75	0
1105	51.75	0.875	3.50	0
1106	51.75	0.875	7.00	0
1107	51.75	0.875	14.00	0
1108	51.75	0.875	17.50	0
1109	51.75	0.875	24.50	0
1110	51.75	0.875	26.25	0
1111	67.50	5.25	10.50	0
1112	67.50	5.25	17.50	0
1113	67.50	-1.00	-2.00	0
1114	67.50	-1.00	30.00	0
1115	67.50	0.00	0.00	0
1116	67.50	0.00	28.00	0
1117	67.50	5.25	14.00	0
1118	67.50	0.00	14.00	0
1119	67.50	0.00	10.50	0

1120	67.50	0.00	17.50	0
1121	67.50	3.50	7.00	0
1122	67.50	1.75	3.50	0
1123	67.50	3.50	21.00	0
1124	67.50	1.75	24.50	0
1125	67.50	0.00	7.00	0
1126	67.50	0.00	3.50	0
1127	67.50	0.00	21.00	0
1128	67.50	0.00	24.50	0
1129	67.50	0.875	26.25	0
1130	67.50	0.875	1.75	0
1131	71.00	-1.00	-2.00	0
1132	71.00	-1.00	30.00	0
1133	71.00	0.00	0.00	0
1134	71.00	0.00	28.00	0
1135	71.00	3.50	7.00	0
1136	71.00	3.50	21.00	0
1137	71.00	3.50	14.00	0
1138	71.00	0.00	14.00	0
1139	71.00	0.00	7.00	0
1140	71.00	0.00	21.00	0
1141	71.00	3.50	10.50	0
1142	71.00	3.50	17.50	0
1143	71.00	0.00	10.50	0
1144	71.00	0.00	17.50	0
1145	71.00	1.75	3.50	0
1146	71.00	1.75	24.50	0
1147	71.00	0.00	3.50	0
1148	71.00	0.00	24.50	0
1149	71.00	0.875	26.25	0
1150	71.00	0.875	1.75	0
1151	74.50	0.00	14.00	0
1152	74.50	-1.00	-2.00	0
1153	74.50	-1.00	30.00	0
1154	74.50	0.00	0.00	0
1155	74.50	0.00	28.00	0
1156	74.50	1.75	3.50	0
1157	74.50	1.75	24.50	0
1158	74.50	1.75	14.00	0
1159	74.50	0.00	3.50	0
1160	74.50	0.00	24.50	0
1161	74.50	1.75	7.00	0
1162	74.50	1.75	10.50	0
1163	74.50	1.75	17.50	0
1164	74.50	1.75	21.00	0
1165	74.50	0.00	7.00	0
1166	74.50	0.00	10.50	0
1167	74.50	0.00	17.50	0
1168	74.50	0.00	21.00	0
1169	74.50	0.875	26.25	0
1170	74.50	0.875	1.75	0
1171	80.00	-1.00	-2.00	0
1172	80.00	-1.00	30.00	0
1173	80.00	-1.00	14.00	0
1174	80.00	-1.00	10.50	0
1175	80.00	-1.00	3.50	0
1176	80.00	-1.00	21.00	0
1177	80.00	-1.00	24.50	0
1178	78.00	0.00	0.00	0
1179	78.00	0.00	28.00	0
1180	78.00	0.00	14.00	0
1181	78.00	0.00	10.50	0

1182	78.00	0.00	3.50	0
1183	78.00	0.00	21.00	0
1184	78.00	0.00	24.50	0
1185	76.25	0.875	1.75	0
1186	76.25	0.875	3.50	0
1187	76.25	0.875	10.50	0
1188	76.25	0.875	14.00	0
1189	76.25	0.875	21.00	0
1190	76.25	0.875	24.50	0
1191	76.25	0.875	26.25	0
1192	50.00	0.00	10.50	0
1193	51.75	0.875	10.50	0
1194	48.00	-1.00	10.50	0
1195	50.00	0.00	21.00	0
1196	51.75	0.875	21.00	0
1197	48.00	-1.00	21.00	0
1198	78.00	0.00	17.50	0
1199	80.00	-1.00	17.50	0
1200	76.25	0.875	17.50	0
1201	78.00	0.00	7.00	0
1202	80.00	-1.00	7.00	0
1203	76.25	0.875	7.00	0
1204	50.1667	0.0833	0.1667	0
1205	50.1667	0.0833	27.8333	0
1206	50.1667	0.0833	14.00	0
1207	50.1667	0.0833	7.00	0
1208	50.1667	0.0833	3.50	0
1209	50.1667	0.0833	17.50	0
1210	50.1667	0.0833	24.50	0
1211	50.1667	0.0833	10.50	0
1212	50.1667	0.0833	21.00	0
1214	64.00	0.0833	0.1667	0
1215	53.50	0.0833	0.1667	0
1216	57.00	0.0833	0.1667	0
1217	60.50	0.0833	0.1667	0
1218	67.50	0.0833	0.1667	0
1219	71.00	0.0833	0.1667	0
1220	74.50	0.0833	0.1667	0
1221	77.8333	0.0833	0.1667	0
1223	64.00	0.0833	27.8333	0
1224	53.50	0.0833	27.8333	0
1225	57.00	0.0833	27.8333	0
1226	60.50	0.0833	27.8333	0
1227	67.50	0.0833	27.8333	0
1228	71.00	0.0833	27.8333	0
1229	74.50	0.0833	27.8333	0
1230	77.8333	0.0833	27.8333	0
1234	77.8333	0.0833	14.00	0
1235	77.8333	0.0833	10.50	0
1236	77.8333	0.0833	3.50	0
1237	77.8333	0.0833	21.00	0
1238	77.8333	0.0833	24.50	0
1239	77.8333	0.0833	17.50	0
1240	77.8333	0.0833	7.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
1016	1	1	1	0	1	0
1017	1	1	0	0	1	0
1018	1	1	1	0	1	0
1019	1	1	1	0	1	0
1020	1	1	0	0	1	0
1024	1	1	1	0	1	0
1025	1	1	0	0	1	0
1026	1	1	1	0	1	0
1028	1	1	1	0	1	0
1032	1	1	1	0	1	0
1034	1	1	1	0	1	0
1037	1	1	1	0	1	0
1038	1	1	0	0	1	0
1047	1	1	1	0	1	0
1048	1	1	0	0	1	0
1115	1	1	1	0	1	0
1116	1	1	0	0	1	0
1133	1	1	1	0	1	0
1134	1	1	0	0	1	0
1154	1	1	1	0	1	0
1155	1	1	0	0	1	0
1178	0	1	1	0	1	0
1179	0	1	0	0	1	0
1180	0	1	1	0	1	0
1181	0	1	1	0	1	0
1182	0	1	1	0	1	0
1183	0	1	1	0	1	0
1184	0	1	1	0	1	0
1192	1	1	1	0	1	0
1195	1	1	1	0	1	0
1198	0	1	1	0	1	0
1201	0	1	1	0	1	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	lg factor
1398	1011	1012	PEAK TRUSS	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1399	1018	1020	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1400	1024	1025	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1401	1039	1022	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1402	1042	1023	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1403	1021	1019	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1404	1039	1042	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1405	1068	1026	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1406	1054	1028	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1407	1070	1032	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1408	1057	1034	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1409	1012	1014	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1410	1012	1015	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1411	1054	1016	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1412	1010	1023	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1413	1009	1029	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1414	1029	1027	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1415	1027	1194	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1416	1013	1031	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1417	1031	1197	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1418	1033	1010	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00

1419	1022	1009	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1420	1037	1038	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1421	1040	1035	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1422	1040	1043	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1423	1043	1036	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1424	1017	1057	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1425	1047	1048	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1426	1041	1045	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1427	1041	1044	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1428	1044	1046	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1429	1051	1021	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1430	1050	1053	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1431	1049	1052	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1432	1039	1054	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1433	1040	1055	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1434	1041	1056	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1435	1044	1059	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1436	1043	1058	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1437	1042	1057	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1438	1062	1068	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1439	1063	1069	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1440	1064	1070	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1441	1065	1071	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1442	1060	1066	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1443	1061	1067	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1444	1074	1080	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1445	1073	1079	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1446	1072	1078	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1447	1075	1081	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1448	1076	1082	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1449	1077	1083	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1450	1085	1091	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1451	1084	1090	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1452	1088	1094	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1453	1089	1095	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1454	1086	1092	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1455	1087	1093	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1456	1064	1021	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1457	1070	1065	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1458	1071	1042	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1459	1057	1096	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1460	1053	1061	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1461	1067	1043	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1462	1058	1087	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1463	1093	1097	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1464	1063	1021	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1465	1062	1069	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1466	1039	1068	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1467	1100	1054	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1468	1053	1060	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1469	1066	1040	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1470	1055	1086	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1471	1092	1101	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1472	1052	1044	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1473	1059	1088	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1474	1094	1089	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1475	1095	1098	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1476	1052	1041	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1477	1056	1084	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1478	1090	1085	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1479	1091	1102	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1480	1011	1072	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00

1481	1078	1073	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1482	1079	1074	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1483	1080	1103	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1484	1011	1075	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1485	1081	1076	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1486	1082	1077	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1487	1083	1099	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1488	1040	1062	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1489	1063	1060	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1490	1060	1041	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1491	1051	1050	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1492	1050	1049	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1493	1064	1061	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1494	1061	1044	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1495	1065	1043	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1496	1049	1012	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1497	1064	1031	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1498	1042	1033	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1499	1051	1013	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1500	1062	1027	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1501	1039	1029	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1502	1103	1102	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1503	1102	1101	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1504	1101	1100	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1505	1100	1104	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1506	1086	1039	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1507	1086	1085	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1508	1085	1074	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1509	1073	1084	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1510	1084	1040	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1512	1014	1045	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1513	1045	1035	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1514	1035	1022	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1515	1110	1109	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1516	1196	1108	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1517	1108	1107	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1518	1193	1106	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1519	1106	1105	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1520	1105	1104	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1521	1110	1096	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1522	1096	1097	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1523	1097	1098	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1524	1098	1099	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1525	1042	1087	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1526	1087	1089	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1527	1089	1077	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1528	1043	1088	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1529	1088	1076	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1530	1044	1075	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1531	1023	1036	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1532	1036	1046	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1533	1015	1046	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1534	1042	1010	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1535	1042	1043	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1536	1044	1043	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1537	1012	1044	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1538	1039	1009	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1539	1039	1040	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1540	1040	1041	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1541	1041	1012	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1542	1115	1116	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1543	1111	1113	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00

1544	1111	1112	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1545	1112	1114	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1546	1117	1118	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1547	1111	1119	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1548	1112	1120	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1549	1122	1126	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1550	1121	1125	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1551	1123	1127	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1552	1124	1128	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1553	1118	1112	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1554	1120	1123	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1555	1127	1124	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1556	1128	1129	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1557	1118	1111	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1558	1119	1121	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1559	1125	1122	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1560	1126	1130	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1561	1133	1134	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1562	1135	1131	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1563	1135	1136	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1564	1136	1132	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1565	1137	1138	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1566	1135	1139	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1567	1136	1140	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1568	1141	1143	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1569	1142	1144	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1570	1145	1147	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1571	1146	1148	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1572	1138	1142	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1573	1144	1136	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1574	1140	1146	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1575	1148	1149	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1576	1138	1141	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1577	1143	1135	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1578	1139	1145	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1579	1147	1150	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1580	1154	1155	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1581	1156	1152	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1582	1157	1153	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1583	1156	1157	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1584	1158	1151	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1585	1156	1159	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1586	1157	1160	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1587	1161	1165	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1588	1162	1166	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1589	1163	1167	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1590	1164	1168	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1591	1163	1151	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1592	1167	1164	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1593	1168	1157	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1594	1160	1169	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1595	1162	1151	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1596	1161	1166	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1597	1156	1165	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1598	1170	1159	BRACE	aisiSSMA_S 400S162-43	A570 Gr50 cold form	0.00	0.00	0.00
1599	1171	1175	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1600	1175	1202	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1601	1174	1173	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1602	1173	1199	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1603	1176	1177	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1604	1177	1172	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1605	1160	1184	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00

1606	1160	1179	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1607	1168	1183	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1608	1151	1180	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1609	1166	1181	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1610	1159	1182	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1611	1159	1178	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1612	1156	1175	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1613	1162	1174	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1614	1158	1173	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1615	1164	1176	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1616	1157	1177	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1617	1156	1171	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1618	1157	1172	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1619	1172	1153	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1620	1153	1132	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1621	1132	1114	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1622	1114	1015	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1623	1171	1152	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1624	1152	1131	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1625	1131	1113	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1626	1113	1014	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1627	1103	1130	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1628	1130	1150	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1629	1150	1170	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1630	1170	1185	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1631	1185	1186	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1632	1203	1187	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1633	1187	1188	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1634	1188	1200	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1635	1189	1190	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1636	1190	1191	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1637	1191	1169	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1638	1169	1149	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1639	1149	1129	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1640	1129	1099	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1641	1157	1136	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1642	1136	1112	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1643	1012	1112	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1644	1156	1135	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1645	1135	1111	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1646	1111	1012	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1647	1164	1136	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1648	1163	1142	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1649	1158	1137	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1650	1142	1112	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1651	1137	1117	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1652	1117	1012	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1653	1162	1141	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1654	1141	1111	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1655	1161	1135	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1656	1157	1146	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1657	1146	1124	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1658	1124	1077	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1659	1136	1123	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1660	1123	1076	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1661	1112	1075	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1662	1156	1145	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1663	1145	1122	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1664	1122	1074	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1665	1135	1121	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1666	1121	1073	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1511	1072	1041	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00

1667	1111	1072	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1668	1054	1092	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1669	1092	1091	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1670	1091	1080	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1671	1080	1126	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1672	1126	1147	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1673	1147	1159	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1674	1068	1055	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1675	1055	1090	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1676	1090	1079	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1677	1079	1125	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1678	1125	1139	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1679	1139	1165	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1680	1069	1066	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1681	1066	1056	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1682	1056	1078	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1683	1078	1119	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1684	1119	1143	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1685	1143	1166	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1686	1021	1053	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1687	1053	1052	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1688	1052	1011	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1689	1011	1118	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1690	1118	1138	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1691	1138	1151	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1692	1070	1067	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1693	1067	1059	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1694	1059	1081	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1695	1081	1120	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1696	1120	1144	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1697	1144	1167	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1698	1071	1058	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1699	1058	1094	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1700	1094	1082	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1701	1082	1127	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1702	1127	1140	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1703	1140	1168	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1704	1057	1093	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1705	1093	1095	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1706	1095	1083	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1707	1083	1128	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1708	1128	1148	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1709	1148	1160	BOTTOM BRACING	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1710	1069	1192	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1711	1063	1194	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1712	1194	1013	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1713	1071	1195	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1714	1065	1197	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1715	1167	1198	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1716	1163	1199	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1717	1165	1201	BOTTOM CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1718	1161	1202	TOP CHORD	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1719	1196	1109	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1720	1203	1186	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1753	1197	1033	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1754	1199	1176	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1755	1202	1174	EAVE MEMBER	aisiSSMA_T 1000T200-54	A570 Gr50 cold form	0.00	0.00	0.00
1756	1200	1189	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1757	1193	1107	PERIMETER MEMBER	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1758	1204	1208	PERIMETER MEMBER AT W.	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1759	1208	1207	PERIMETER MEMBER AT W.	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1760	1207	1211	PERIMETER MEMBER AT W.	aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00

1761	1211	1206	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1762	1206	1209	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1763	1209	1212	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1764	1212	1210	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1765	1210	1205	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1766	1221	1220	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1767	1220	1219	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1768	1219	1218	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1769	1218	1214	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1770	1214	1217	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1771	1217	1216	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1772	1216	1215	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1773	1215	1204	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1774	1205	1224	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1775	1224	1225	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1776	1225	1226	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1777	1226	1223	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1778	1223	1227	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1779	1227	1228	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1780	1228	1229	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1781	1229	1230	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1782	1230	1238	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1783	1238	1237	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1784	1237	1239	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1785	1239	1234	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1786	1234	1235	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1787	1235	1240	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1788	1240	1236	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00
1789	1236	1221	PERIMETER MEMBER AT W.aisiSSMA_S 1000S250-97	A570 Gr50 cold form	0.00	0.00	0.00

Orientation of local axes

Member	Rotation [Deg]	Axes23	NX	NY	NZ
1398	90.00	0	0.00	0.00	0.00
1401	180.00	0	0.00	0.00	0.00
1409	180.00	0	0.00	0.00	0.00
1421	180.00	0	0.00	0.00	0.00
1426	180.00	0	0.00	0.00	0.00
1429	90.00	0	0.00	0.00	0.00
1430	90.00	0	0.00	0.00	0.00
1431	90.00	0	0.00	0.00	0.00
1432	90.00	0	0.00	0.00	0.00
1433	90.00	0	0.00	0.00	0.00
1434	90.00	0	0.00	0.00	0.00
1435	90.00	0	0.00	0.00	0.00
1436	90.00	0	0.00	0.00	0.00
1437	90.00	0	0.00	0.00	0.00
1438	90.00	0	0.00	0.00	0.00
1439	90.00	0	0.00	0.00	0.00
1440	90.00	0	0.00	0.00	0.00
1441	90.00	0	0.00	0.00	0.00
1442	90.00	0	0.00	0.00	0.00
1443	90.00	0	0.00	0.00	0.00
1444	90.00	0	0.00	0.00	0.00
1445	90.00	0	0.00	0.00	0.00
1446	90.00	0	0.00	0.00	0.00
1447	90.00	0	0.00	0.00	0.00

1448	90.00	0	0.00	0.00	0.00
1449	90.00	0	0.00	0.00	0.00
1450	90.00	0	0.00	0.00	0.00
1451	90.00	0	0.00	0.00	0.00
1452	90.00	0	0.00	0.00	0.00
1453	90.00	0	0.00	0.00	0.00
1454	90.00	0	0.00	0.00	0.00
1455	90.00	0	0.00	0.00	0.00
1543	180.00	0	0.00	0.00	0.00
1546	90.00	0	0.00	0.00	0.00
1547	90.00	0	0.00	0.00	0.00
1548	90.00	0	0.00	0.00	0.00
1549	90.00	0	0.00	0.00	0.00
1550	90.00	0	0.00	0.00	0.00
1551	90.00	0	0.00	0.00	0.00
1552	90.00	0	0.00	0.00	0.00
1562	180.00	0	0.00	0.00	0.00
1565	90.00	0	0.00	0.00	0.00
1566	90.00	0	0.00	0.00	0.00
1567	90.00	0	0.00	0.00	0.00
1568	90.00	0	0.00	0.00	0.00
1569	90.00	0	0.00	0.00	0.00
1570	90.00	0	0.00	0.00	0.00
1571	90.00	0	0.00	0.00	0.00
1581	180.00	0	0.00	0.00	0.00
1584	90.00	0	0.00	0.00	0.00
1585	90.00	0	0.00	0.00	0.00
1586	90.00	0	0.00	0.00	0.00
1587	90.00	0	0.00	0.00	0.00
1588	90.00	0	0.00	0.00	0.00
1589	90.00	0	0.00	0.00	0.00
1590	90.00	0	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	**M33**	**M22**	**V3**	**V2**	**M33**	**M22**	**V3**	**V2**	**TOR**	**AXL**	**Axial rigidity**
1398	1	1	0	0	1	1	0	0	0	0	Full
1403	1	1	0	0	0	0	0	0	0	0	Full
1404	1	1	0	0	1	1	0	0	0	0	Full
1405	1	1	0	0	0	0	0	0	0	0	Full
1406	1	1	0	0	0	0	0	0	0	0	Full
1407	1	1	0	0	0	0	0	0	0	0	Full
1408	1	1	0	0	0	0	0	0	0	0	Full
1411	1	1	0	0	0	0	0	0	0	0	Full
1412	1	1	0	0	1	1	0	0	0	0	Full
1413	1	1	0	0	1	1	0	0	0	0	Full
1414	1	1	0	0	1	1	0	0	0	0	Full
1415	1	1	0	0	1	1	0	0	0	0	Full
1416	1	1	0	0	1	1	0	0	0	0	Full
1417	1	1	0	0	1	1	0	0	0	0	Full
1418	1	1	0	0	1	1	0	0	0	0	Full
1419	1	1	0	0	1	1	0	0	0	0	Full
1422	1	1	0	0	1	1	0	0	0	0	Full
1424	0	0	0	0	1	1	0	0	0	0	Full
1427	1	1	0	0	1	1	0	0	0	0	Full
1429	1	1	0	0	1	1	0	0	0	0	Full

1430	1	1	0	0	1	1	0	0	0	0	Full
1431	1	1	0	0	1	1	0	0	0	0	Full
1432	1	1	0	0	1	1	0	0	0	0	Full
1433	1	1	0	0	1	1	0	0	0	0	Full
1434	1	1	0	0	1	1	0	0	0	0	Full
1435	1	1	0	0	1	1	0	0	0	0	Full
1436	1	1	0	0	1	1	0	0	0	0	Full
1437	1	1	0	0	1	1	0	0	0	0	Full
1438	1	1	0	0	1	1	0	0	0	0	Full
1439	1	1	0	0	1	1	0	0	0	0	Full
1440	1	1	0	0	1	1	0	0	0	0	Full
1441	1	1	0	0	1	1	0	0	0	0	Full
1442	1	1	0	0	1	1	0	0	0	0	Full
1443	1	1	0	0	1	1	0	0	0	0	Full
1444	1	1	0	0	1	1	0	0	0	0	Full
1445	1	1	0	0	1	1	0	0	0	0	Full
1446	1	1	0	0	1	1	0	0	0	0	Full
1447	1	1	0	0	1	1	0	0	0	0	Full
1448	1	1	0	0	1	1	0	0	0	0	Full
1449	1	1	0	0	1	1	0	0	0	0	Full
1450	1	1	0	0	1	1	0	0	0	0	Full
1451	1	1	0	0	1	1	0	0	0	0	Full
1452	1	1	0	0	1	1	0	0	0	0	Full
1453	1	1	0	0	1	1	0	0	0	0	Full
1454	1	1	0	0	1	1	0	0	0	0	Full
1455	1	1	0	0	1	1	0	0	0	0	Full
1456	1	1	0	0	1	1	0	0	0	0	Full
1457	1	1	0	0	1	1	0	0	0	0	Full
1458	1	1	0	0	1	1	0	0	0	0	Full
1459	1	1	0	0	1	1	0	0	0	0	Full
1460	1	1	0	0	1	1	0	0	0	0	Full
1461	1	1	0	0	1	1	0	0	0	0	Full
1462	1	1	0	0	1	1	0	0	0	0	Full
1463	1	1	0	0	1	1	0	0	0	0	Full
1464	1	1	0	0	1	1	0	0	0	0	Full
1465	1	1	0	0	1	1	0	0	0	0	Full
1466	1	1	0	0	1	1	0	0	0	0	Full
1467	1	1	0	0	1	1	0	0	0	0	Full
1468	1	1	0	0	1	1	0	0	0	0	Full
1469	1	1	0	0	1	1	0	0	0	0	Full
1470	1	1	0	0	1	1	0	0	0	0	Full
1471	1	1	0	0	1	1	0	0	0	0	Full
1472	1	1	0	0	1	1	0	0	0	0	Full
1473	1	1	0	0	1	1	0	0	0	0	Full
1474	1	1	0	0	1	1	0	0	0	0	Full
1475	1	1	0	0	1	1	0	0	0	0	Full
1476	1	1	0	0	1	1	0	0	0	0	Full
1477	1	1	0	0	1	1	0	0	0	0	Full
1478	1	1	0	0	1	1	0	0	0	0	Full
1479	1	1	0	0	1	1	0	0	0	0	Full
1480	1	1	0	0	1	1	0	0	0	0	Full
1481	1	1	0	0	1	1	0	0	0	0	Full
1482	1	1	0	0	1	1	0	0	0	0	Full
1483	1	1	0	0	1	1	0	0	0	0	Full
1484	1	1	0	0	1	1	0	0	0	0	Full
1485	1	1	0	0	1	1	0	0	0	0	Full
1486	1	1	0	0	1	1	0	0	0	0	Full
1487	1	1	0	0	1	1	0	0	0	0	Full
1488	1	1	0	0	1	1	0	0	0	0	Full
1489	1	1	0	0	1	1	0	0	0	0	Full
1490	1	1	0	0	1	1	0	0	0	0	Full

1491	1	1	0	0	1	1	0	0	0	0	Full
1492	1	1	0	0	1	1	0	0	0	0	Full
1493	1	1	0	0	1	1	0	0	0	0	Full
1494	1	1	0	0	1	1	0	0	0	0	Full
1495	1	1	0	0	1	1	0	0	0	0	Full
1496	1	1	0	0	1	1	0	0	0	0	Full
1497	1	1	0	0	0	0	0	0	0	0	Full
1498	1	1	0	0	0	0	0	0	0	0	Full
1499	1	1	0	0	0	0	0	0	0	0	Full
1500	1	1	0	0	0	0	0	0	0	0	Full
1501	1	1	0	0	0	0	0	0	0	0	Full
1502	1	1	0	0	1	1	0	0	0	0	Full
1503	1	1	0	0	1	1	0	0	0	0	Full
1504	1	1	0	0	1	1	0	0	0	0	Full
1505	1	1	0	0	1	1	0	0	0	0	Full
1506	1	1	0	0	1	1	0	0	0	0	Full
1507	1	1	0	0	1	1	0	0	0	0	Full
1508	1	1	0	0	1	1	0	0	0	0	Full
1509	1	1	0	0	1	1	0	0	0	0	Full
1510	1	1	0	0	1	1	0	0	0	0	Full
1512	1	1	0	0	1	1	0	0	0	0	Full
1513	1	1	0	0	1	1	0	0	0	0	Full
1514	1	1	0	0	1	1	0	0	0	0	Full
1515	1	1	0	0	1	1	0	0	0	0	Full
1516	1	1	0	0	1	1	0	0	0	0	Full
1517	1	1	0	0	1	1	0	0	0	0	Full
1518	1	1	0	0	1	1	0	0	0	0	Full
1519	1	1	0	0	1	1	0	0	0	0	Full
1520	1	1	0	0	1	1	0	0	0	0	Full
1521	1	1	0	0	1	1	0	0	0	0	Full
1522	1	1	0	0	1	1	0	0	0	0	Full
1523	1	1	0	0	1	1	0	0	0	0	Full
1524	1	1	0	0	1	1	0	0	0	0	Full
1525	1	1	0	0	1	1	0	0	0	0	Full
1526	1	1	0	0	1	1	0	0	0	0	Full
1527	1	1	0	0	1	1	0	0	0	0	Full
1528	1	1	0	0	1	1	0	0	0	0	Full
1529	1	1	0	0	1	1	0	0	0	0	Full
1530	1	1	0	0	1	1	0	0	0	0	Full
1531	1	1	0	0	1	1	0	0	0	0	Full
1532	1	1	0	0	1	1	0	0	0	0	Full
1533	1	1	0	0	1	1	0	0	0	0	Full
1534	1	1	0	0	0	0	0	0	0	0	Full
1535	1	1	0	0	1	1	0	0	0	0	Full
1536	1	1	0	0	1	1	0	0	0	0	Full
1537	1	1	0	0	1	1	0	0	0	0	Full
1538	1	1	0	0	0	0	0	0	0	0	Full
1539	1	1	0	0	1	1	0	0	0	0	Full
1540	1	1	0	0	1	1	0	0	0	0	Full
1541	1	1	0	0	1	1	0	0	0	0	Full
1544	1	1	0	0	1	1	0	0	0	0	Full
1546	1	1	0	0	1	1	0	0	0	0	Full
1547	1	1	0	0	1	1	0	0	0	0	Full
1548	1	1	0	0	1	1	0	0	0	0	Full
1549	1	1	0	0	1	1	0	0	0	0	Full
1550	1	1	0	0	1	1	0	0	0	0	Full
1551	1	1	0	0	1	1	0	0	0	0	Full
1552	1	1	0	0	1	1	0	0	0	0	Full
1553	1	1	0	0	1	1	0	0	0	0	Full
1554	1	1	0	0	1	1	0	0	0	0	Full
1555	1	1	0	0	1	1	0	0	0	0	Full

1556	1	1	0	0	1	1	0	0	0	0	Full
1557	1	1	0	0	1	1	0	0	0	0	Full
1558	1	1	0	0	1	1	0	0	0	0	Full
1559	1	1	0	0	1	1	0	0	0	0	Full
1560	1	1	0	0	1	1	0	0	0	0	Full
1563	1	1	0	0	1	1	0	0	0	0	Full
1565	1	1	0	0	1	1	0	0	0	0	Full
1566	1	1	0	0	1	1	0	0	0	0	Full
1567	1	1	0	0	1	1	0	0	0	0	Full
1568	1	1	0	0	1	1	0	0	0	0	Full
1569	1	1	0	0	1	1	0	0	0	0	Full
1570	1	1	0	0	1	1	0	0	0	0	Full
1571	1	1	0	0	1	1	0	0	0	0	Full
1572	1	1	0	0	1	1	0	0	0	0	Full
1573	1	1	0	0	1	1	0	0	0	0	Full
1574	1	1	0	0	1	1	0	0	0	0	Full
1575	1	1	0	0	1	1	0	0	0	0	Full
1576	1	1	0	0	1	1	0	0	0	0	Full
1577	1	1	0	0	1	1	0	0	0	0	Full
1578	1	1	0	0	1	1	0	0	0	0	Full
1579	1	1	0	0	1	1	0	0	0	0	Full
1583	1	1	0	0	1	1	0	0	0	0	Full
1584	1	1	0	0	1	1	0	0	0	0	Full
1585	1	1	0	0	1	1	0	0	0	0	Full
1586	1	1	0	0	1	1	0	0	0	0	Full
1587	1	1	0	0	1	1	0	0	0	0	Full
1588	1	1	0	0	1	1	0	0	0	0	Full
1589	1	1	0	0	1	1	0	0	0	0	Full
1590	1	1	0	0	1	1	0	0	0	0	Full
1591	1	1	0	0	1	1	0	0	0	0	Full
1592	1	1	0	0	1	1	0	0	0	0	Full
1593	1	1	0	0	1	1	0	0	0	0	Full
1594	1	1	0	0	1	1	0	0	0	0	Full
1595	1	1	0	0	1	1	0	0	0	0	Full
1596	1	1	0	0	1	1	0	0	0	0	Full
1597	1	1	0	0	1	1	0	0	0	0	Full
1598	1	1	0	0	1	1	0	0	0	0	Full
1599	1	1	0	0	1	1	0	0	0	0	Full
1600	1	1	0	0	1	1	0	0	0	0	Full
1601	1	1	0	0	1	1	0	0	0	0	Full
1602	1	1	0	0	1	1	0	0	0	0	Full
1603	1	1	0	0	1	1	0	0	0	0	Full
1604	1	1	0	0	1	1	0	0	0	0	Full
1605	1	1	0	0	0	0	0	0	0	0	Full
1606	1	1	0	0	0	0	0	0	0	0	Full
1607	1	1	0	0	0	0	0	0	0	0	Full
1608	1	1	0	0	0	0	0	0	0	0	Full
1609	1	1	0	0	0	0	0	0	0	0	Full
1610	1	1	0	0	0	0	0	0	0	0	Full
1611	1	1	0	0	0	0	0	0	0	0	Full
1617	1	1	0	0	0	0	0	0	0	0	Full
1618	1	1	0	0	0	0	0	0	0	0	Full
1619	1	1	0	0	1	1	0	0	0	0	Full
1620	1	1	0	0	1	1	0	0	0	0	Full
1621	1	1	0	0	1	1	0	0	0	0	Full
1622	1	1	0	0	1	1	0	0	0	0	Full
1623	1	1	0	0	1	1	0	0	0	0	Full
1624	1	1	0	0	1	1	0	0	0	0	Full
1625	1	1	0	0	1	1	0	0	0	0	Full
1626	1	1	0	0	1	1	0	0	0	0	Full
1627	1	1	0	0	1	1	0	0	0	0	Full

1628	1	1	0	0	1	1	0	0	0	0	Full
1629	1	1	0	0	1	1	0	0	0	0	Full
1630	1	1	0	0	1	1	0	0	0	0	Full
1631	1	1	0	0	1	1	0	0	0	0	Full
1632	1	1	0	0	1	1	0	0	0	0	Full
1633	1	1	0	0	1	1	0	0	0	0	Full
1634	1	1	0	0	1	1	0	0	0	0	Full
1635	1	1	0	0	1	1	0	0	0	0	Full
1636	1	1	0	0	1	1	0	0	0	0	Full
1637	1	1	0	0	1	1	0	0	0	0	Full
1638	1	1	0	0	1	1	0	0	0	0	Full
1639	1	1	0	0	1	1	0	0	0	0	Full
1640	1	1	0	0	1	1	0	0	0	0	Full
1641	1	1	0	0	1	1	0	0	0	0	Full
1642	1	1	0	0	1	1	0	0	0	0	Full
1643	1	1	0	0	1	1	0	0	0	0	Full
1644	1	1	0	0	1	1	0	0	0	0	Full
1645	1	1	0	0	1	1	0	0	0	0	Full
1646	1	1	0	0	1	1	0	0	0	0	Full
1647	1	1	0	0	1	1	0	0	0	0	Full
1648	1	1	0	0	1	1	0	0	0	0	Full
1649	1	1	0	0	1	1	0	0	0	0	Full
1650	1	1	0	0	1	1	0	0	0	0	Full
1651	1	1	0	0	1	1	0	0	0	0	Full
1652	1	1	0	0	1	1	0	0	0	0	Full
1653	1	1	0	0	1	1	0	0	0	0	Full
1654	1	1	0	0	1	1	0	0	0	0	Full
1655	1	1	0	0	1	1	0	0	0	0	Full
1656	1	1	0	0	1	1	0	0	0	0	Full
1657	1	1	0	0	1	1	0	0	0	0	Full
1658	1	1	0	0	1	1	0	0	0	0	Full
1659	1	1	0	0	1	1	0	0	0	0	Full
1660	1	1	0	0	1	1	0	0	0	0	Full
1661	1	1	0	0	1	1	0	0	0	0	Full
1662	1	1	0	0	1	1	0	0	0	0	Full
1663	1	1	0	0	1	1	0	0	0	0	Full
1664	1	1	0	0	1	1	0	0	0	0	Full
1665	1	1	0	0	1	1	0	0	0	0	Full
1666	1	1	0	0	1	1	0	0	0	0	Full
1511	1	1	0	0	1	1	0	0	0	0	Full
1667	1	1	0	0	1	1	0	0	0	0	Full
1668	1	1	0	0	1	1	0	0	0	0	Full
1669	1	1	0	0	1	1	0	0	0	0	Full
1670	1	1	0	0	1	1	0	0	0	0	Full
1671	1	1	0	0	1	1	0	0	0	0	Full
1672	1	1	0	0	1	1	0	0	0	0	Full
1673	1	1	0	0	1	1	0	0	0	0	Full
1674	1	1	0	0	1	1	0	0	0	0	Full
1675	1	1	0	0	1	1	0	0	0	0	Full
1676	1	1	0	0	1	1	0	0	0	0	Full
1677	1	1	0	0	1	1	0	0	0	0	Full
1678	1	1	0	0	1	1	0	0	0	0	Full
1679	1	1	0	0	1	1	0	0	0	0	Full
1680	1	1	0	0	1	1	0	0	0	0	Full
1681	1	1	0	0	1	1	0	0	0	0	Full
1682	1	1	0	0	1	1	0	0	0	0	Full
1683	1	1	0	0	1	1	0	0	0	0	Full
1684	1	1	0	0	1	1	0	0	0	0	Full
1685	1	1	0	0	1	1	0	0	0	0	Full
1686	1	1	0	0	1	1	0	0	0	0	Full
1687	1	1	0	0	1	1	0	0	0	0	Full

1688	1	1	0	0	1	1	0	0	0	0	Full
1689	1	1	0	0	1	1	0	0	0	0	Full
1690	1	1	0	0	1	1	0	0	0	0	Full
1691	1	1	0	0	1	1	0	0	0	0	Full
1692	1	1	0	0	1	1	0	0	0	0	Full
1693	1	1	0	0	1	1	0	0	0	0	Full
1694	1	1	0	0	1	1	0	0	0	0	Full
1695	1	1	0	0	1	1	0	0	0	0	Full
1696	1	1	0	0	1	1	0	0	0	0	Full
1697	1	1	0	0	1	1	0	0	0	0	Full
1698	1	1	0	0	1	1	0	0	0	0	Full
1699	1	1	0	0	1	1	0	0	0	0	Full
1700	1	1	0	0	1	1	0	0	0	0	Full
1701	1	1	0	0	1	1	0	0	0	0	Full
1702	1	1	0	0	1	1	0	0	0	0	Full
1703	1	1	0	0	1	1	0	0	0	0	Full
1704	1	1	0	0	1	1	0	0	0	0	Full
1705	1	1	0	0	1	1	0	0	0	0	Full
1706	1	1	0	0	1	1	0	0	0	0	Full
1707	1	1	0	0	1	1	0	0	0	0	Full
1708	1	1	0	0	1	1	0	0	0	0	Full
1709	1	1	0	0	1	1	0	0	0	0	Full
1710	1	1	0	0	0	0	0	0	0	0	Full
1711	1	1	0	0	0	0	0	0	0	0	Full
1712	1	1	0	0	1	1	0	0	0	0	Full
1713	1	1	0	0	0	0	0	0	0	0	Full
1714	1	1	0	0	0	0	0	0	0	0	Full
1715	1	1	0	0	0	0	0	0	0	0	Full
1717	1	1	0	0	0	0	0	0	0	0	Full
1719	1	1	0	0	1	1	0	0	0	0	Full
1753	1	1	0	0	1	1	0	0	0	0	Full
1754	1	1	0	0	1	1	0	0	0	0	Full
1755	1	1	0	0	1	1	0	0	0	0	Full
1756	1	1	0	0	1	1	0	0	0	0	Full
1757	1	1	0	0	1	1	0	0	0	0	Full
1758	1	1	0	0	1	1	0	0	0	0	Full
1759	1	1	0	0	1	1	0	0	0	0	Full
1760	1	1	0	0	1	1	0	0	0	0	Full
1761	1	1	0	0	1	1	0	0	0	0	Full
1762	1	1	0	0	1	1	0	0	0	0	Full
1763	1	1	0	0	1	1	0	0	0	0	Full
1764	1	1	0	0	1	1	0	0	0	0	Full
1765	1	1	0	0	1	1	0	0	0	0	Full
1766	1	1	0	0	1	1	0	0	0	0	Full
1767	1	1	0	0	1	1	0	0	0	0	Full
1768	1	1	0	0	1	1	0	0	0	0	Full
1769	1	1	0	0	1	1	0	0	0	0	Full
1770	1	1	0	0	1	1	0	0	0	0	Full
1771	1	1	0	0	1	1	0	0	0	0	Full
1772	1	1	0	0	1	1	0	0	0	0	Full
1773	1	1	0	0	1	1	0	0	0	0	Full
1774	1	1	0	0	1	1	0	0	0	0	Full
1775	1	1	0	0	1	1	0	0	0	0	Full
1776	1	1	0	0	1	1	0	0	0	0	Full
1777	1	1	0	0	1	1	0	0	0	0	Full
1778	1	1	0	0	1	1	0	0	0	0	Full
1779	1	1	0	0	1	1	0	0	0	0	Full
1780	1	1	0	0	1	1	0	0	0	0	Full
1781	1	1	0	0	1	1	0	0	0	0	Full
1782	1	1	0	0	1	1	0	0	0	0	Full
1783	1	1	0	0	1	1	0	0	0	0	Full

1784	1	1	0	0	1	1	0	0	0	0	Full
1785	1	1	0	0	1	1	0	0	0	0	Full
1786	1	1	0	0	1	1	0	0	0	0	Full
1787	1	1	0	0	1	1	0	0	0	0	Full
1788	1	1	0	0	1	1	0	0	0	0	Full
1789	1	1	0	0	1	1	0	0	0	0	Full

Glossary

Cb22, Cb33	: Moment gradient coefficients
Cm22, Cm33	: Coefficients applied to bending term in interaction formula
d0	: Tapered member section depth at J end of member
DJX	: Rigid end offset distance measured from J node in axis X
DJY	: Rigid end offset distance measured from J node in axis Y
DJZ	: Rigid end offset distance measured from J node in axis Z
DKX	: Rigid end offset distance measured from K node in axis X
DKY	: Rigid end offset distance measured from K node in axis Y
DKZ	: Rigid end offset distance measured from K node in axis Z
dL	: Tapered member section depth at K end of member
Ig factor	: Inertia reduction factor (Effective Inertia/Gross Inertia) for reinforced concrete members
K22	: Effective length factor about axis 2
K33	: Effective length factor about axis 3
L22	: Member length for calculation of axial capacity
L33	: Member length for calculation of axial capacity
LB pos	: Lateral unbraced length of the compression flange in the positive side of local axis 2
LB neg	: Lateral unbraced length of the compression flange in the negative side of local axis 2
RX	: Rotation about X
RY	: Rotation about Y
RZ	: Rotation about Z
TO	: 1 = Tension only member 0 = Normal member
TX	: Translation in X
TY	: Translation in Y
TZ	: Translation in Z

Current Date: 9/15/2026 5:46 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx

Load data

Load Conditions

Condition	Description	Comb.	Category
D1	DL	Yes	
D2	DL+LL	Yes	
D3	DL+SL	Yes	
D4	DL+0.75LL	Yes	
D5	DL+0.75SL	Yes	
D6	DL+0.6Wx	Yes	
D7	DL+0.6Wz	Yes	
D8	DL+0.45Wx+0.75LL	Yes	
D9	DL+0.45Wz+0.75LL	Yes	
D10	DL+0.45Wx+0.75SL	Yes	
D11	DL+0.45Wz+0.75SL	Yes	
D12	0.6DL+0.6Wx	Yes	
D13	0.6DL+0.6Wz	Yes	
D14	DL+0.7EQx	Yes	
D15	DL+0.7EQz	Yes	
D16	DL+0.525EQx	Yes	
D17	DL+0.525EQz	Yes	
D18	DL+0.75SL	Yes	
D19	DL+0.525EQx+0.75SL	Yes	
D20	DL+0.525EQz+0.75SL	Yes	
D21	0.6DL+0.7EQx	Yes	
D22	0.6DL+0.7EQz	Yes	
S1	DL	Yes	
S2	DL+LL	Yes	
S3	DL+SL	Yes	
S4	DL+0.75LL	Yes	
S5	DL+0.75SL	Yes	
S6	DL+0.6Wx	Yes	
S7	DL+0.6Wz	Yes	
S8	DL+0.7EQx	Yes	
S9	DL+0.7EQz	Yes	
S10	DL+0.45Wx+0.75LL	Yes	
S11	DL+0.45Wz+0.75LL	Yes	
S12	DL+0.45Wx+0.75SL	Yes	
S13	DL+0.45Wz+0.75SL	Yes	
S14	0.6DL+0.6Wx	Yes	
S15	0.6DL+0.6Wz	Yes	
S16	DL+0.7EQx	Yes	
S17	DL+0.7EQz	Yes	
S18	DL+0.525EQx	Yes	
S19	DL+0.525EQz	Yes	
S20	DL+0.75SL	Yes	
S21	DL+0.525EQx+0.75SL	Yes	
S22	DL+0.525EQz+0.75SL	Yes	
S23	0.6DL+0.7EQx	Yes	
S24	0.6DL+0.7EQz	Yes	

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier			
		Comb.	MultX	MultY	MultZ
D1	DL	Yes	0.00	0.00	0.00
D2	DL+LL	Yes	0.00	0.00	0.00
D3	DL+SL	Yes	0.00	0.00	0.00
D4	DL+0.75LL	Yes	0.00	0.00	0.00
D5	DL+0.75SL	Yes	0.00	0.00	0.00
D6	DL+0.6Wx	Yes	0.00	0.00	0.00
D7	DL+0.6Wz	Yes	0.00	0.00	0.00
D8	DL+0.45Wx+0.75LL	Yes	0.00	0.00	0.00
D9	DL+0.45Wz+0.75LL	Yes	0.00	0.00	0.00
D10	DL+0.45Wx+0.75SL	Yes	0.00	0.00	0.00
D11	DL+0.45Wz+0.75SL	Yes	0.00	0.00	0.00
D12	0.6DL+0.6Wx	Yes	0.00	0.00	0.00
D13	0.6DL+0.6Wz	Yes	0.00	0.00	0.00
D14	DL+0.7EQx	Yes	0.00	0.00	0.00
D15	DL+0.7EQz	Yes	0.00	0.00	0.00
D16	DL+0.525EQx	Yes	0.00	0.00	0.00
D17	DL+0.525EQz	Yes	0.00	0.00	0.00
D18	DL+0.75SL	Yes	0.00	0.00	0.00
D19	DL+0.525EQx+0.75SL	Yes	0.00	0.00	0.00
D20	DL+0.525EQz+0.75SL	Yes	0.00	0.00	0.00
D21	0.6DL+0.7EQx	Yes	0.00	0.00	0.00
D22	0.6DL+0.7EQz	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+LL	Yes	0.00	0.00	0.00
S3	DL+SL	Yes	0.00	0.00	0.00
S4	DL+0.75LL	Yes	0.00	0.00	0.00
S5	DL+0.75SL	Yes	0.00	0.00	0.00
S6	DL+0.6Wx	Yes	0.00	0.00	0.00
S7	DL+0.6Wz	Yes	0.00	0.00	0.00
S8	DL+0.7EQx	Yes	0.00	0.00	0.00
S9	DL+0.7EQz	Yes	0.00	0.00	0.00
S10	DL+0.45Wx+0.75LL	Yes	0.00	0.00	0.00
S11	DL+0.45Wz+0.75LL	Yes	0.00	0.00	0.00
S12	DL+0.45Wx+0.75SL	Yes	0.00	0.00	0.00
S13	DL+0.45Wz+0.75SL	Yes	0.00	0.00	0.00
S14	0.6DL+0.6Wx	Yes	0.00	0.00	0.00
S15	0.6DL+0.6Wz	Yes	0.00	0.00	0.00
S16	DL+0.7EQx	Yes	0.00	0.00	0.00
S17	DL+0.7EQz	Yes	0.00	0.00	0.00
S18	DL+0.525EQx	Yes	0.00	0.00	0.00
S19	DL+0.525EQz	Yes	0.00	0.00	0.00
S20	DL+0.75SL	Yes	0.00	0.00	0.00
S21	DL+0.525EQx+0.75SL	Yes	0.00	0.00	0.00
S22	DL+0.525EQz+0.75SL	Yes	0.00	0.00	0.00
S23	0.6DL+0.7EQx	Yes	0.00	0.00	0.00
S24	0.6DL+0.7EQz	Yes	0.00	0.00	0.00

Earthquake (Dynamic analysis only)

Condition	a/g	Ang. [Deg]	Damp. [%]
D1	0.00	0.00	0.00
D2	0.00	0.00	0.00
D3	0.00	0.00	0.00
D4	0.00	0.00	0.00
D5	0.00	0.00	0.00

D6	0.00	0.00	0.00
D7	0.00	0.00	0.00
D8	0.00	0.00	0.00
D9	0.00	0.00	0.00
D10	0.00	0.00	0.00
D11	0.00	0.00	0.00
D12	0.00	0.00	0.00
D13	0.00	0.00	0.00
D14	0.00	0.00	0.00
D15	0.00	0.00	0.00
D16	0.00	0.00	0.00
D17	0.00	0.00	0.00
D18	0.00	0.00	0.00
D19	0.00	0.00	0.00
D20	0.00	0.00	0.00
D21	0.00	0.00	0.00
D22	0.00	0.00	0.00
S1	0.00	0.00	0.00
S2	0.00	0.00	0.00
S3	0.00	0.00	0.00
S4	0.00	0.00	0.00
S5	0.00	0.00	0.00
S6	0.00	0.00	0.00
S7	0.00	0.00	0.00
S8	0.00	0.00	0.00
S9	0.00	0.00	0.00
S10	0.00	0.00	0.00
S11	0.00	0.00	0.00
S12	0.00	0.00	0.00
S13	0.00	0.00	0.00
S14	0.00	0.00	0.00
S15	0.00	0.00	0.00
S16	0.00	0.00	0.00
S17	0.00	0.00	0.00
S18	0.00	0.00	0.00
S19	0.00	0.00	0.00
S20	0.00	0.00	0.00
S21	0.00	0.00	0.00
S22	0.00	0.00	0.00
S23	0.00	0.00	0.00
S24	0.00	0.00	0.00

Seismic Response Spectra

Glossary

Comb : Indicates if load condition is a load combination

Current Date: 9/15/2026 5:47 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx

Analysis result

Nodes

Nodal displacements envelope

Note.- **Ic** is the controlling load condition
Nodal displacements envelope for :

D1=DL
D2=DL+LL
D3=DL+SL
D4=DL+0.75LL
D5=DL+0.75SL
D6=DL+0.6Wx
D7=DL+0.6Wz
D8=DL+0.45Wx+0.75LL
D9=DL+0.45Wz+0.75LL
D10=DL+0.45Wx+0.75SL
D11=DL+0.45Wz+0.75SL
D12=0.6DL+0.6Wx
D13=0.6DL+0.6Wz
D14=DL+0.7EQx
D15=DL+0.7EQz
D16=DL+0.525EQx
D17=DL+0.525EQz
D18=DL+0.75SL
D19=DL+0.525EQx+0.75SL
D20=DL+0.525EQz+0.75SL
D21=0.6DL+0.7EQx
D22=0.6DL+0.7EQz
S1=DL
S2=DL+LL
S3=DL+SL
S4=DL+0.75LL
S5=DL+0.75SL
S6=DL+0.6Wx
S7=DL+0.6Wz
S8=DL+0.7EQx
S9=DL+0.7EQz
S10=DL+0.45Wx+0.75LL
S11=DL+0.45Wz+0.75LL
S12=DL+0.45Wx+0.75SL
S13=DL+0.45Wz+0.75SL
S14=0.6DL+0.6Wx
S15=0.6DL+0.6Wz
S16=DL+0.7EQx
S17=DL+0.7EQz
S18=DL+0.525EQx
S19=DL+0.525EQz
S20=DL+0.75SL
S21=DL+0.525EQx+0.75SL
S22=DL+0.525EQz+0.75SL
S23=0.6DL+0.7EQx
S24=0.6DL+0.7EQz

Node		Translation						Rotation					
		X [in]	lc	Y [in]	lc	Z [in]	lc	Rx [Rad]	lc	Ry [Rad]	lc	Rz [Rad]	lc
1009	Max	0.001	D12	0.008	D7	0.000	D12	0.00020	D7	0.00006	D12	0.00011	D12
	Min	-0.001	D9	-0.001	D12	-0.003	D7	0.00001	D22	-0.00005	D7	-0.00012	D7
1010	Max	0.003	D12	0.003	D2	0.003	D8	0.00007	D13	-0.00002	D12	0.00043	D8
	Min	-0.001	D9	-0.003	D12	-0.001	D13	-0.00039	D2	-0.00031	D2	0.00002	D13
1011	Max	0.004	D2	-0.005	D13	0.005	D2	0.00014	D13	0.00002	D7	0.00029	D12
	Min	-0.001	D12	-0.030	D2	-0.002	D13	0.00000	D15	0.00000	D12	-0.00012	D2
1012	Max	0.005	D2	-0.003	D13	0.005	D2	0.00011	D13	0.00001	D7	0.00001	D9
	Min	-0.002	D12	-0.021	D2	0.000	D13	0.00000	D8	-0.00001	D12	-0.00017	D12
1013	Max	0.007	D12	0.063	D2	0.001	D8	0.00014	D7	0.00010	D2	0.00058	D12
	Min	-0.032	D2	-0.014	D12	-0.002	D13	-0.00007	D12	-0.00004	D13	-0.00263	D2
1014	Max	0.002	D6	0.013	D2	0.000	D13	0.00053	D2	0.00007	D12	0.00021	D12
	Min	0.000	D22	0.001	D13	-0.006	D2	0.00004	D13	-0.00011	D2	-0.00006	D2
1015	Max	0.005	D6	0.013	D2	0.016	D2	-0.00013	D12	0.00015	D2	0.00040	D12
	Min	0.000	D13	0.003	D12	0.001	D13	-0.00053	D2	-0.00002	D12	-0.00007	D2
1016	Max	0.000	D1	0.000	D1	0.000	D1	0.00022	D9	0.00000	D1	0.00017	D12
	Min	0.000	D1	0.000	D1	0.000	D1	0.00005	D12	0.00000	D1	-0.00022	D2
1017	Max	0.000	D1	0.000	D1	0.002	D2	-0.00003	D13	0.00000	D1	0.00043	D8
	Min	0.000	D1	0.000	D1	0.000	D12	-0.00068	D2	0.00000	D1	-0.00005	D13
1018	Max	0.000	D1	0.000	D1	0.000	D1	0.00058	D2	0.00000	D1	0.00025	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00002	D13	0.00000	D1	-0.00012	D2
1019	Max	0.000	D1	0.000	D1	0.000	D1	0.00014	D9	0.00000	D1	0.00064	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00007	D12	0.00000	D1	-0.00268	D2
1020	Max	0.000	D1	0.000	D1	0.010	D2	-0.00014	D12	0.00000	D1	0.00041	D12
	Min	0.000	D1	0.000	D1	-0.001	D13	-0.00058	D2	0.00000	D1	-0.00015	D2
1021	Max	0.001	D2	0.028	D12	0.004	D2	0.00002	D2	0.00000	D7	0.00036	D12
	Min	0.000	D12	-0.113	D2	-0.001	D12	-0.00002	D12	0.00000	D12	-0.00026	D2
1022	Max	0.001	D12	0.013	D9	0.002	D12	0.00053	D9	0.00008	D12	0.00017	D12
	Min	0.000	D9	-0.004	D12	-0.006	D9	-0.00019	D12	-0.00011	D2	-0.00016	D2
1023	Max	0.004	D12	0.014	D2	0.017	D2	0.00026	D12	0.00003	D9	0.00037	D12
	Min	0.000	D7	-0.006	D12	-0.006	D12	-0.00059	D2	-0.00005	D12	-0.00007	D9
1024	Max	0.000	D1	0.000	D1	0.000	D1	0.00056	D2	0.00000	D1	0.00021	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00017	D12	0.00000	D1	-0.00021	D2
1025	Max	0.000	D1	0.000	D1	0.009	D2	0.00025	D12	0.00000	D1	0.00039	D12
	Min	0.000	D1	0.000	D1	-0.003	D12	-0.00065	D2	0.00000	D1	-0.00009	D9
1026	Max	0.000	D1	0.000	D1	0.000	D1	0.00018	D9	0.00000	D1	0.00037	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00003	D12	0.00000	D1	-0.00160	D2
1027	Max	0.004	D12	0.037	D2	0.001	D6	0.00017	D9	0.00008	D2	0.00031	D12
	Min	-0.019	D2	-0.008	D12	-0.002	D13	-0.00004	D12	-0.00004	D13	-0.00156	D2
1028	Max	0.000	D1	0.000	D1	0.000	D1	0.00022	D9	0.00000	D1	0.00013	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00001	D12	0.00000	D1	-0.00057	D2
1029	Max	0.001	D12	0.012	D2	0.000	D12	0.00021	D9	0.00009	D2	0.00006	D12

	Min	-0.006	D2	-0.002	D12	-0.003	D7	-0.00002	D12	-0.00004	D13	-0.00051	D2
1030	Max	0.001	D2	0.008	D12	0.000	D12	0.00089	D2	0.00002	D2	0.00028	D12
	Min	0.000	D13	-0.032	D2	-0.001	D9	-0.00021	D12	0.00000	D13	-0.00036	D2
1031	Max	0.006	D12	0.057	D2	0.002	D8	0.00013	D7	0.00013	D2	0.00053	D12
	Min	-0.028	D2	-0.013	D12	-0.002	D13	-0.00009	D12	-0.00003	D13	-0.00236	D2
1032	Max	0.000	D1	0.000	D1	0.000	D1	0.00015	D9	0.00000	D1	0.00059	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00009	D12	0.00000	D1	-0.00241	D2
1033	Max	0.002	D12	0.014	D2	0.003	D8	0.00025	D2	0.00036	D2	0.00012	D12
	Min	-0.007	D2	-0.003	D12	-0.002	D13	-0.00012	D12	-0.00001	D13	-0.00057	D2
1034	Max	0.000	D1	0.000	D1	0.000	D1	0.00044	D2	0.00000	D1	0.00019	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00012	D12	0.00000	D1	-0.00063	D2
1035	Max	0.001	D6	0.011	D2	0.002	D12	0.00045	D2	0.00007	D12	0.00018	D12
	Min	0.000	D13	-0.003	D12	-0.005	D2	-0.00015	D12	-0.00010	D2	-0.00011	D2
1036	Max	0.004	D6	0.011	D2	0.016	D2	0.00019	D12	0.00009	D2	0.00038	D12
	Min	0.000	D13	-0.004	D12	-0.006	D12	-0.00044	D2	-0.00004	D12	-0.00009	D9
1037	Max	0.000	D1	0.000	D1	0.000	D1	0.00050	D2	0.00000	D1	0.00022	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00013	D12	0.00000	D1	-0.00017	D2
1038	Max	0.000	D1	0.000	D1	0.011	D2	0.00017	D12	0.00000	D1	0.00040	D12
	Min	0.000	D1	0.000	D1	-0.004	D12	-0.00049	D2	0.00000	D1	-0.00013	D9
1039	Max	0.008	D2	0.007	D12	0.010	D2	0.00054	D2	0.00010	D6	0.00020	D12
	Min	-0.003	D12	-0.026	D2	-0.003	D12	-0.00016	D12	0.00000	D21	-0.00036	D2
1040	Max	0.008	D2	0.010	D12	0.009	D2	0.00011	D9	0.00006	D8	0.00019	D8
	Min	-0.006	D12	-0.028	D2	-0.003	D12	-0.00012	D12	0.00000	D13	0.00003	D22
1041	Max	0.007	D2	0.004	D12	0.006	D2	0.00007	D13	0.00012	D8	0.00019	D2
	Min	-0.004	D12	-0.024	D2	-0.001	D12	-0.00016	D2	0.00001	D13	0.00003	D12
1042	Max	0.010	D2	0.010	D12	0.001	D12	0.00020	D12	-0.00001	D13	0.00033	D12
	Min	-0.005	D12	-0.028	D2	-0.003	D9	-0.00059	D2	-0.00018	D8	-0.00030	D2
1043	Max	0.009	D2	0.013	D12	0.001	D8	0.00014	D12	-0.00001	D21	0.00028	D8
	Min	-0.008	D12	-0.028	D2	-0.001	D13	-0.00010	D2	-0.00007	D8	0.00003	D13
1044	Max	0.007	D2	0.004	D12	0.003	D2	0.00019	D9	-0.00002	D13	0.00019	D2
	Min	-0.005	D12	-0.024	D2	-0.001	D13	0.00004	D22	-0.00018	D8	0.00005	D12
1045	Max	0.001	D6	0.012	D2	0.000	D12	0.00048	D2	0.00007	D12	0.00020	D12
	Min	0.000	D22	0.000	D12	-0.006	D2	-0.00001	D12	-0.00010	D2	-0.00008	D2
1046	Max	0.004	D6	0.012	D2	0.016	D2	0.00001	D12	0.00012	D2	0.00039	D12
	Min	0.000	D13	0.000	D12	-0.001	D12	-0.00048	D2	-0.00003	D12	-0.00008	D2
1047	Max	0.000	D1	0.000	D1	0.000	D1	0.00053	D2	0.00000	D1	0.00023	D12
	Min	0.000	D1	0.000	D1	0.000	D1	0.00001	D12	0.00000	D1	-0.00014	D2
1048	Max	0.000	D1	0.000	D1	0.010	D2	0.00000	D12	0.00000	D1	0.00040	D12
	Min	0.000	D1	0.000	D1	-0.001	D12	-0.00053	D2	0.00000	D1	-0.00014	D2
1049	Max	0.010	D2	0.005	D12	0.005	D2	0.00006	D13	0.00001	D7	0.00019	D2
	Min	-0.006	D12	-0.029	D2	-0.001	D13	-0.00001	D6	-0.00001	D12	0.00004	D12

1050	Max	0.022	D2	0.020	D12	0.005	D2	0.00002	D13	0.00000	D21	0.00023	D8
	Min	-0.012	D12	-0.050	D2	-0.001	D12	-0.00001	D6	-0.00001	D6	0.00003	D13
1051	Max	0.055	D2	0.028	D12	0.004	D2	0.00002	D2	0.00001	D2	0.00027	D12
	Min	-0.015	D12	-0.114	D2	-0.001	D12	-0.00002	D12	-0.00001	D12	-0.00031	D2
1052	Max	0.003	D2	0.004	D12	0.005	D2	0.00010	D13	0.00001	D7	0.00029	D12
	Min	0.000	D12	-0.027	D2	-0.001	D13	-0.00001	D6	0.00000	D12	-0.00012	D2
1053	Max	0.002	D2	0.019	D12	0.005	D2	0.00003	D7	0.00001	D7	0.00029	D12
	Min	0.000	D12	-0.048	D2	-0.002	D12	-0.00002	D12	0.00000	D12	-0.00013	D2
1054	Max	0.001	D2	0.007	D12	0.000	D12	0.00082	D2	0.00003	D2	0.00028	D12
	Min	0.000	D12	-0.026	D2	0.000	D9	-0.00020	D12	0.00001	D13	-0.00036	D2
1055	Max	0.003	D2	0.010	D12	0.002	D2	0.00025	D2	0.00000	D21	0.00025	D12
	Min	0.000	D13	-0.029	D2	0.000	D12	-0.00013	D12	-0.00001	D6	-0.00015	D2
1056	Max	0.004	D2	0.003	D12	0.004	D2	0.00007	D13	0.00000	D13	0.00027	D12
	Min	0.000	D12	-0.028	D2	-0.001	D13	-0.00006	D8	-0.00003	D8	-0.00012	D2
1057	Max	0.002	D2	0.010	D12	0.008	D2	0.00023	D12	0.00000	D12	0.00046	D12
	Min	-0.001	D12	-0.029	D2	-0.002	D12	-0.00085	D2	-0.00003	D2	-0.00015	D9
1058	Max	0.003	D2	0.012	D12	0.008	D2	0.00013	D12	0.00002	D2	0.00034	D12
	Min	0.000	D12	-0.029	D2	-0.003	D12	-0.00026	D2	-0.00001	D12	-0.00013	D2
1059	Max	0.004	D2	0.003	D12	0.006	D2	0.00008	D7	0.00002	D8	0.00031	D12
	Min	0.000	D12	-0.028	D2	-0.001	D13	0.00001	D22	0.00000	D21	-0.00012	D2
1060	Max	0.018	D2	0.017	D12	0.007	D2	0.00028	D2	0.00018	D2	0.00021	D8
	Min	-0.010	D12	-0.044	D2	-0.002	D12	-0.00013	D12	-0.00007	D12	0.00003	D13
1061	Max	0.018	D2	0.018	D12	0.003	D2	0.00010	D12	0.00003	D12	0.00026	D8
	Min	-0.012	D12	-0.044	D2	-0.001	D13	-0.00029	D2	-0.00018	D2	0.00003	D13
1062	Max	0.032	D2	0.017	D12	0.009	D2	0.00094	D2	0.00051	D2	0.00023	D12
	Min	-0.009	D12	-0.069	D2	-0.002	D12	-0.00022	D12	-0.00012	D12	-0.00034	D2
1063	Max	0.049	D2	0.025	D12	0.007	D2	0.00054	D2	0.00029	D2	0.00025	D12
	Min	-0.013	D12	-0.101	D2	-0.002	D12	-0.00014	D12	-0.00008	D12	-0.00032	D2
1064	Max	0.050	D2	0.026	D12	0.001	D2	0.00011	D12	0.00006	D12	0.00029	D12
	Min	-0.014	D12	-0.103	D2	0.000	D13	-0.00052	D2	-0.00027	D2	-0.00030	D2
1065	Max	0.033	D2	0.020	D12	0.001	D12	0.00020	D12	0.00011	D12	0.00031	D12
	Min	-0.011	D12	-0.072	D2	-0.002	D9	-0.00092	D2	-0.00050	D2	-0.00030	D2
1066	Max	0.003	D2	0.016	D12	0.003	D2	0.00026	D2	0.00000	D13	0.00027	D12
	Min	0.000	D12	-0.041	D2	-0.001	D12	-0.00012	D12	-0.00002	D8	-0.00014	D2
1067	Max	0.002	D2	0.017	D12	0.007	D2	0.00009	D12	0.00001	D8	0.00031	D12
	Min	0.000	D12	-0.041	D2	-0.002	D12	-0.00026	D2	0.00000	D21	-0.00013	D2
1068	Max	0.002	D2	0.017	D12	0.000	D12	0.00096	D2	0.00000	D2	0.00030	D12
	Min	0.000	D13	-0.067	D2	-0.001	D9	-0.00022	D12	0.00000	D12	-0.00033	D2
1069	Max	0.001	D2	0.024	D12	0.001	D2	0.00056	D2	0.00000	D13	0.00033	D12
	Min	0.000	D12	-0.100	D2	0.000	D12	-0.00014	D12	-0.00001	D8	-0.00030	D2

1070	Max	0.001	D2	0.026	D12	0.006	D2	0.00011	D12	0.00001	D12	0.00039	D12
	Min	0.000	D12	-0.102	D2	-0.001	D12	-0.00053	D2	0.00000	D2	-0.00023	D9
1071	Max	0.001	D2	0.019	D12	0.008	D2	0.00020	D12	0.00002	D2	0.00042	D12
	Min	0.000	D12	-0.070	D2	-0.002	D12	-0.00094	D2	-0.00001	D12	-0.00019	D9
1072	Max	0.005	D2	-0.001	D13	0.010	D2	0.00005	D13	0.00002	D12	0.00000	D13
	Min	-0.005	D12	-0.031	D2	-0.001	D13	-0.00023	D2	-0.00001	D2	-0.00008	D6
1073	Max	0.005	D2	0.002	D13	0.012	D2	0.00008	D2	0.00001	D12	0.00003	D12
	Min	-0.006	D12	-0.032	D2	-0.001	D13	0.00002	D12	-0.00002	D2	-0.00004	D2
1074	Max	0.004	D2	0.002	D13	0.009	D2	0.00037	D2	0.00002	D6	0.00014	D12
	Min	-0.005	D12	-0.023	D2	-0.001	D13	-0.00003	D13	0.00000	D22	-0.00009	D2
1075	Max	0.005	D2	-0.007	D12	0.000	D22	0.00025	D9	0.00001	D9	0.00000	D13
	Min	-0.006	D12	-0.031	D2	-0.003	D7	0.00007	D22	-0.00008	D12	-0.00007	D6
1076	Max	0.005	D2	-0.008	D12	-0.001	D22	0.00002	D13	0.00004	D2	0.00009	D12
	Min	-0.009	D12	-0.032	D2	-0.006	D7	-0.00008	D2	-0.00006	D12	-0.00005	D2
1077	Max	0.005	D2	-0.006	D12	0.000	D2	-0.00009	D12	0.00001	D2	0.00026	D12
	Min	-0.007	D12	-0.023	D2	-0.005	D13	-0.00037	D2	-0.00002	D12	-0.00010	D2
1078	Max	0.005	D2	0.000	D13	0.004	D2	0.00008	D13	0.00000	D13	0.00027	D12
	Min	0.000	D12	-0.034	D2	-0.002	D13	-0.00007	D2	-0.00004	D8	-0.00012	D2
1079	Max	0.006	D2	0.002	D13	0.003	D2	0.00012	D2	0.00000	D21	0.00026	D12
	Min	0.001	D13	-0.033	D2	-0.001	D13	0.00002	D13	-0.00003	D6	-0.00012	D2
1080	Max	0.004	D2	0.002	D13	0.001	D2	0.00041	D2	0.00011	D2	0.00025	D12
	Min	0.001	D13	-0.023	D2	-0.001	D13	-0.00003	D13	0.00002	D13	-0.00012	D2
1081	Max	0.005	D2	-0.008	D12	0.006	D2	0.00012	D9	0.00002	D9	0.00032	D12
	Min	0.000	D12	-0.034	D2	-0.002	D13	0.00002	D22	0.00001	D21	-0.00013	D2
1082	Max	0.006	D2	-0.008	D12	0.007	D2	-0.00002	D13	0.00001	D2	0.00034	D12
	Min	0.000	D12	-0.033	D2	-0.002	D13	-0.00012	D2	0.00000	D12	-0.00013	D2
1083	Max	0.004	D2	-0.006	D12	0.008	D2	-0.00010	D12	0.00000	D12	0.00038	D12
	Min	0.000	D12	-0.022	D2	-0.001	D13	-0.00041	D2	-0.00010	D2	-0.00014	D2
1084	Max	0.006	D2	0.001	D12	0.010	D2	0.00003	D7	0.00006	D8	0.00013	D8
	Min	-0.006	D12	-0.028	D2	-0.001	D13	-0.00004	D12	0.00000	D13	0.00002	D13
1085	Max	0.005	D2	0.001	D13	0.008	D2	0.00031	D2	0.00007	D8	0.00016	D12
	Min	-0.004	D12	-0.021	D2	-0.001	D13	-0.00002	D13	0.00001	D13	-0.00004	D2
1086	Max	0.006	D2	0.005	D12	0.007	D2	0.00030	D2	0.00010	D8	0.00017	D12
	Min	-0.004	D12	-0.020	D2	-0.002	D12	-0.00010	D12	0.00002	D13	-0.00006	D2
1087	Max	0.008	D2	0.007	D12	0.003	D2	0.00013	D12	-0.00003	D13	0.00029	D12
	Min	-0.006	D12	-0.019	D2	-0.001	D12	-0.00031	D2	-0.00015	D8	-0.00005	D9
1088	Max	0.007	D2	0.001	D12	0.001	D2	0.00005	D12	-0.00001	D21	0.00019	D8
	Min	-0.009	D12	-0.027	D2	-0.003	D13	-0.00001	D2	-0.00010	D6	0.00003	D13
1089	Max	0.006	D2	0.000	D12	0.002	D2	0.00001	D12	-0.00002	D13	0.00028	D12
	Min	-0.007	D12	-0.020	D2	-0.003	D13	-0.00031	D2	-0.00009	D8	-0.00005	D2
1090	Max	0.005	D2	0.001	D12	0.003	D2	0.00007	D2	0.00000	D21	0.00025	D12

	Min	0.001	D13	-0.029	D2	-0.001	D13	-0.00004	D12	-0.00002	D6	-0.00013	D2
1091	Max	0.003	D2	0.001	D13	0.001	D2	0.00036	D2	0.00008	D2	0.00024	D12
	Min	0.001	D13	-0.020	D2	0.000	D13	-0.00002	D13	0.00002	D13	-0.00013	D2
1092	Max	0.002	D2	0.005	D12	0.001	D2	0.00034	D2	0.00005	D2	0.00024	D12
	Min	0.001	D13	-0.019	D2	0.000	D12	-0.00011	D12	0.00001	D13	-0.00016	D2
1093	Max	0.003	D2	0.007	D12	0.010	D2	0.00015	D12	0.00000	D12	0.00037	D12
	Min	0.000	D12	-0.019	D2	-0.004	D12	-0.00034	D2	-0.00006	D2	-0.00013	D2
1094	Max	0.004	D2	0.001	D12	0.007	D2	0.00004	D12	0.00002	D2	0.00034	D12
	Min	0.000	D12	-0.028	D2	-0.001	D12	-0.00007	D2	-0.00001	D12	-0.00013	D2
1095	Max	0.004	D2	0.000	D12	0.009	D2	0.00001	D12	0.00000	D12	0.00037	D12
	Min	0.000	D12	-0.020	D2	-0.001	D12	-0.00035	D2	-0.00008	D2	-0.00013	D2
1096	Max	0.004	D2	0.005	D12	0.003	D2	0.00023	D12	-0.00002	D13	0.00037	D12
	Min	-0.004	D12	-0.015	D2	0.000	D12	-0.00066	D2	-0.00016	D2	-0.00023	D9
1097	Max	0.004	D2	0.004	D12	0.006	D2	0.00016	D12	-0.00003	D12	0.00035	D12
	Min	-0.004	D12	-0.011	D2	-0.003	D12	-0.00045	D2	-0.00016	D2	-0.00013	D2
1098	Max	0.003	D2	0.000	D12	0.005	D2	0.00000	D12	-0.00002	D12	0.00035	D12
	Min	-0.004	D12	-0.012	D2	-0.002	D13	-0.00048	D2	-0.00012	D2	-0.00013	D2
1099	Max	0.003	D2	-0.003	D12	0.004	D2	-0.00013	D12	0.00001	D12	0.00034	D12
	Min	-0.004	D12	-0.013	D2	-0.003	D13	-0.00053	D2	-0.00006	D2	-0.00015	D2
1100	Max	0.004	D2	0.004	D12	0.005	D2	0.00059	D2	0.00005	D2	0.00022	D12
	Min	-0.003	D12	-0.013	D2	-0.001	D12	-0.00016	D12	0.00001	D13	-0.00031	D2
1101	Max	0.003	D2	0.003	D12	0.004	D2	0.00046	D2	0.00011	D2	0.00021	D12
	Min	-0.003	D12	-0.011	D2	-0.001	D12	-0.00012	D12	0.00000	D12	-0.00015	D2
1102	Max	0.003	D2	0.001	D13	0.005	D2	0.00048	D2	0.00009	D2	0.00021	D12
	Min	-0.003	D12	-0.012	D2	0.000	D13	-0.00003	D13	-0.00001	D12	-0.00012	D2
1103	Max	0.002	D2	0.001	D13	0.006	D2	0.00053	D2	0.00005	D2	0.00021	D12
	Min	-0.003	D12	-0.013	D2	-0.001	D13	-0.00005	D13	-0.00002	D12	-0.00012	D2
1104	Max	0.004	D2	0.003	D12	0.004	D2	0.00036	D2	0.00001	D12	0.00014	D12
	Min	-0.003	D12	-0.011	D2	0.000	D12	-0.00005	D12	-0.00001	D2	-0.00030	D2
1105	Max	0.004	D2	0.003	D12	0.003	D2	0.00039	D2	0.00002	D12	0.00017	D12
	Min	-0.002	D12	-0.013	D2	0.000	D12	-0.00007	D12	-0.00006	D2	-0.00056	D2
1106	Max	0.016	D2	0.009	D12	0.003	D2	0.00062	D2	0.00010	D9	0.00042	D12
	Min	-0.004	D12	-0.035	D2	0.000	D12	-0.00014	D12	-0.00002	D12	-0.00163	D2
1107	Max	0.028	D2	0.014	D12	0.002	D2	0.00009	D9	0.00002	D13	0.00069	D12
	Min	-0.008	D12	-0.057	D2	0.000	D12	-0.00004	D12	-0.00006	D2	-0.00269	D2
1108	Max	0.025	D2	0.013	D12	0.002	D2	0.00002	D12	0.00001	D12	0.00064	D12
	Min	-0.007	D12	-0.052	D2	-0.001	D12	-0.00020	D2	-0.00013	D2	-0.00243	D2
1109	Max	0.005	D2	0.005	D12	0.001	D2	0.00002	D12	0.00006	D2	0.00024	D12
	Min	-0.003	D12	-0.015	D2	-0.001	D12	-0.00014	D2	-0.00002	D12	-0.00063	D2
1110	Max	0.004	D2	0.005	D12	0.000	D2	0.00002	D12	0.00000	D12	0.00023	D12
	Min	-0.004	D12	-0.012	D2	-0.001	D12	-0.00049	D2	-0.00023	D2	-0.00020	D9

1111	Max	0.003	D2	-0.001	D13	0.007	D8	0.00006	D13	0.00001	D12	-0.00005	D13
	Min	-0.005	D12	-0.026	D8	-0.001	D13	-0.00015	D2	-0.00014	D2	-0.00027	D2
1112	Max	0.003	D2	-0.007	D13	0.003	D2	0.00019	D9	0.00013	D2	-0.00006	D12
	Min	-0.007	D12	-0.026	D8	-0.002	D13	0.00003	D12	-0.00008	D12	-0.00029	D2
1113	Max	0.002	D8	0.012	D8	-0.001	D13	0.00049	D8	0.00007	D12	0.00023	D12
	Min	0.000	D22	0.001	D13	-0.006	D8	0.00006	D13	-0.00014	D2	-0.00006	D2
1114	Max	0.005	D6	0.012	D8	0.017	D8	-0.00014	D21	0.00021	D2	0.00040	D12
	Min	0.001	D13	0.004	D21	0.002	D13	-0.00048	D8	-0.00001	D12	-0.00009	D2
1115	Max	0.000	D1	0.000	D1	0.000	D1	0.00054	D2	0.00000	D1	0.00027	D12
	Min	0.000	D1	0.000	D1	0.000	D1	0.00000	D13	0.00000	D1	-0.00012	D2
1116	Max	0.000	D1	0.000	D1	0.011	D8	-0.00016	D21	0.00000	D1	0.00040	D12
	Min	0.000	D1	0.000	D1	0.000	D13	-0.00053	D2	0.00000	D1	-0.00019	D2
1117	Max	0.000	D21	-0.005	D13	0.005	D2	0.00007	D13	0.00000	D13	-0.00006	D13
	Min	-0.009	D6	-0.031	D8	-0.001	D13	0.00000	D2	-0.00002	D6	-0.00028	D2
1118	Max	0.006	D2	-0.005	D13	0.005	D8	0.00010	D13	0.00002	D13	0.00030	D12
	Min	-0.001	D12	-0.029	D8	-0.002	D13	0.00000	D2	-0.00001	D6	-0.00015	D2
1119	Max	0.006	D2	-0.001	D13	0.004	D8	0.00007	D13	0.00000	D13	0.00028	D12
	Min	0.000	D12	-0.029	D8	-0.001	D13	-0.00003	D2	-0.00005	D8	-0.00014	D2
1120	Max	0.006	D2	-0.008	D21	0.006	D8	0.00009	D7	0.00003	D9	0.00032	D12
	Min	-0.001	D12	-0.029	D8	-0.002	D13	-0.00001	D12	0.00001	D21	-0.00016	D2
1121	Max	0.003	D2	0.001	D13	0.010	D2	0.00003	D7	0.00001	D12	0.00004	D12
	Min	-0.007	D12	-0.028	D8	-0.001	D13	0.00000	D21	-0.00010	D2	-0.00021	D2
1122	Max	0.003	D2	0.001	D13	0.008	D2	0.00032	D2	0.00002	D12	0.00015	D12
	Min	-0.005	D12	-0.021	D2	-0.001	D13	-0.00003	D13	-0.00006	D2	-0.00016	D2
1123	Max	0.003	D2	-0.008	D21	0.001	D8	0.00004	D13	0.00012	D2	0.00009	D12
	Min	-0.010	D12	-0.028	D8	-0.004	D13	-0.00003	D6	-0.00006	D12	-0.00024	D2
1124	Max	0.003	D2	-0.006	D21	0.003	D8	-0.00009	D21	0.00010	D2	0.00026	D12
	Min	-0.008	D12	-0.021	D2	-0.004	D13	-0.00031	D2	-0.00001	D12	-0.00019	D2
1125	Max	0.008	D2	0.001	D13	0.003	D2	0.00008	D8	0.00000	D21	0.00027	D12
	Min	0.001	D13	-0.029	D8	-0.001	D13	0.00002	D21	-0.00004	D6	-0.00014	D2
1126	Max	0.005	D2	0.001	D13	0.001	D2	0.00036	D2	0.00014	D2	0.00027	D12
	Min	0.001	D13	-0.020	D2	0.000	D13	-0.00002	D13	0.00003	D13	-0.00013	D2
1127	Max	0.007	D2	-0.008	D21	0.008	D8	0.00000	D13	0.00001	D2	0.00034	D12
	Min	0.000	D12	-0.028	D8	-0.001	D13	-0.00008	D8	0.00000	D12	-0.00017	D2
1128	Max	0.005	D2	-0.006	D21	0.009	D8	-0.00011	D21	0.00000	D12	0.00037	D12
	Min	0.000	D12	-0.020	D2	-0.001	D13	-0.00035	D2	-0.00013	D2	-0.00018	D2
1129	Max	0.003	D2	-0.004	D21	0.006	D8	-0.00014	D21	0.00003	D8	0.00034	D12
	Min	-0.005	D12	-0.012	D2	-0.002	D13	-0.00048	D2	0.00000	D13	-0.00021	D2
1130	Max	0.002	D2	0.001	D13	0.005	D2	0.00048	D2	0.00000	D13	0.00022	D12
	Min	-0.003	D12	-0.012	D2	-0.001	D13	-0.00004	D13	-0.00002	D6	-0.00015	D2

1131	Max	0.003	D8	0.014	D8	-0.002	D13	0.00058	D8	0.00008	D12	0.00025	D12
	Min	0.001	D22	0.003	D13	-0.007	D8	0.00014	D13	-0.00021	D2	-0.00010	D2
1132	Max	0.005	D6	0.014	D8	0.021	D8	-0.00013	D13	0.00034	D2	0.00038	D12
	Min	0.001	D22	0.003	D13	0.003	D13	-0.00057	D8	0.00003	D12	-0.00021	D2
1133	Max	0.000	D1	0.000	D1	0.000	D1	0.00062	D8	0.00000	D1	0.00029	D12
	Min	0.000	D1	0.000	D1	0.000	D1	0.00008	D13	0.00000	D1	-0.00021	D2
1134	Max	0.000	D1	0.000	D1	0.014	D8	-0.00015	D21	0.00000	D1	0.00036	D12
	Min	0.000	D1	0.000	D1	0.001	D13	-0.00061	D8	0.00000	D1	-0.00038	D2
1135	Max	0.001	D2	-0.004	D13	0.012	D8	0.00018	D8	0.00002	D12	0.00011	D12
	Min	-0.007	D12	-0.036	D8	0.001	D13	0.00003	D21	-0.00006	D2	-0.00015	D2
1136	Max	0.000	D9	-0.009	D21	0.000	D6	0.00001	D13	0.00011	D2	0.00013	D12
	Min	-0.011	D12	-0.036	D8	-0.002	D13	-0.00019	D8	-0.00004	D12	-0.00019	D2
1137	Max	-0.004	D21	-0.013	D13	0.006	D8	0.00002	D13	0.00003	D7	0.00012	D12
	Min	-0.025	D8	-0.063	D8	0.000	D13	0.00000	D2	-0.00001	D12	-0.00017	D2
1138	Max	0.007	D2	-0.012	D13	0.007	D8	0.00003	D13	0.00002	D13	0.00031	D12
	Min	-0.001	D12	-0.061	D8	0.000	D13	0.00000	D2	-0.00001	D6	-0.00025	D2
1139	Max	0.009	D2	-0.004	D13	0.003	D8	0.00034	D8	0.00000	D21	0.00029	D12
	Min	0.001	D13	-0.038	D8	0.000	D13	0.00008	D21	-0.00004	D6	-0.00022	D2
1140	Max	0.008	D2	-0.009	D21	0.011	D8	-0.00005	D13	0.00001	D2	0.00033	D12
	Min	0.000	D12	-0.038	D8	0.000	D13	-0.00034	D8	0.00000	D12	-0.00030	D2
1141	Max	-0.003	D21	-0.010	D13	0.009	D8	0.00035	D8	-0.00006	D21	0.00012	D12
	Min	-0.019	D8	-0.056	D8	0.000	D13	0.00009	D21	-0.00025	D8	-0.00016	D2
1142	Max	-0.002	D13	-0.012	D13	0.003	D8	-0.00006	D13	0.00022	D8	0.00012	D12
	Min	-0.019	D8	-0.056	D8	-0.001	D13	-0.00035	D8	0.00006	D21	-0.00019	D2
1143	Max	0.008	D2	-0.009	D13	0.004	D8	0.00033	D8	0.00000	D13	0.00030	D12
	Min	0.000	D12	-0.053	D8	-0.001	D13	0.00008	D21	-0.00005	D8	-0.00023	D2
1144	Max	0.007	D2	-0.012	D13	0.009	D8	-0.00005	D13	0.00003	D9	0.00032	D12
	Min	-0.001	D12	-0.053	D8	0.000	D13	-0.00032	D8	0.00001	D21	-0.00027	D2
1145	Max	0.001	D2	-0.002	D13	0.009	D8	0.00039	D8	0.00004	D12	0.00019	D12
	Min	-0.005	D12	-0.024	D8	0.000	D13	0.00004	D13	-0.00009	D2	-0.00015	D2
1146	Max	0.001	D2	-0.006	D21	0.004	D8	-0.00009	D21	0.00016	D2	0.00025	D12
	Min	-0.008	D12	-0.024	D8	-0.002	D13	-0.00040	D8	-0.00001	D12	-0.00023	D2
1147	Max	0.007	D2	-0.002	D13	0.001	D8	0.00044	D8	0.00017	D2	0.00029	D12
	Min	0.002	D13	-0.024	D8	0.000	D13	0.00004	D13	0.00003	D13	-0.00021	D2
1148	Max	0.006	D2	-0.006	D21	0.012	D8	-0.00011	D21	0.00000	D12	0.00035	D12
	Min	0.000	D12	-0.024	D8	0.001	D13	-0.00044	D8	-0.00015	D2	-0.00033	D2
1149	Max	0.002	D2	-0.003	D21	0.008	D8	-0.00014	D21	0.00014	D2	0.00033	D12
	Min	-0.005	D12	-0.014	D8	-0.001	D13	-0.00056	D8	0.00003	D13	-0.00028	D2
1150	Max	0.001	D2	-0.001	D13	0.006	D8	0.00056	D8	-0.00001	D12	0.00026	D12
	Min	-0.003	D12	-0.014	D8	0.000	D13	0.00004	D13	-0.00007	D2	-0.00017	D2
1151	Max	0.008	D2	-0.035	D21	0.005	D8	0.00000	D12	0.00003	D13	0.00072	D6

	Min	-0.001	D12	-0.140	D8	0.001	D13	0.00000	D2	-0.00001	D6	0.00008	D21
1152	Max	0.003	D8	0.019	D8	-0.002	D21	0.00078	D8	0.00011	D12	0.00030	D12
	Min	0.001	D22	0.004	D21	-0.009	D8	0.00018	D21	-0.00041	D2	-0.00031	D2
1153	Max	0.006	D8	0.019	D8	0.020	D8	-0.00013	D13	0.00073	D2	0.00028	D12
	Min	0.001	D22	0.003	D13	0.003	D13	-0.00078	D8	0.00013	D12	-0.00061	D2
1154	Max	0.000	D1	0.000	D1	0.000	D1	0.00083	D8	0.00000	D1	0.00035	D12
	Min	0.000	D1	0.000	D1	0.000	D1	0.00020	D21	0.00000	D1	-0.00052	D2
1155	Max	0.000	D1	0.000	D1	0.011	D8	-0.00018	D13	0.00000	D1	0.00022	D12
	Min	0.000	D1	0.000	D1	0.001	D13	-0.00084	D8	0.00000	D1	-0.00097	D2
1156	Max	0.000	D21	-0.008	D13	0.015	D8	0.00074	D8	0.00028	D8	0.00078	D8
	Min	-0.006	D6	-0.036	D8	0.003	D13	0.00017	D13	0.00007	D21	0.00018	D13
1157	Max	0.000	D21	-0.008	D13	-0.001	D22	-0.00017	D13	-0.00003	D21	0.00078	D8
	Min	-0.009	D6	-0.036	D8	-0.005	D8	-0.00073	D8	-0.00011	D2	0.00018	D13
1158	Max	-0.014	D21	-0.035	D21	0.005	D8	0.00000	D12	0.00003	D7	0.00331	D8
	Min	-0.064	D8	-0.141	D8	0.001	D13	0.00000	D2	0.00000	D22	0.00083	D21
1159	Max	0.008	D2	-0.009	D13	0.001	D8	0.00105	D8	0.00019	D2	0.00050	D6
	Min	0.002	D13	-0.036	D8	0.000	D13	0.00025	D13	0.00004	D13	0.00006	D21
1160	Max	0.007	D2	-0.009	D13	0.009	D8	-0.00026	D21	0.00000	D12	0.00051	D12
	Min	0.000	D12	-0.036	D8	0.001	D13	-0.00104	D8	-0.00018	D2	-0.00030	D2
1161	Max	-0.007	D21	-0.022	D13	0.012	D8	0.00113	D8	-0.00010	D21	0.00206	D8
	Min	-0.033	D8	-0.089	D8	0.003	D13	0.00029	D21	-0.00042	D8	0.00051	D21
1162	Max	-0.012	D21	-0.032	D21	0.009	D8	0.00064	D8	-0.00008	D13	0.00298	D8
	Min	-0.055	D8	-0.127	D8	0.002	D13	0.00016	D21	-0.00032	D8	0.00074	D21
1163	Max	-0.012	D21	-0.032	D21	0.001	D6	-0.00016	D21	0.00032	D8	0.00297	D8
	Min	-0.056	D8	-0.127	D8	0.000	D13	-0.00065	D8	0.00008	D21	0.00074	D21
1164	Max	-0.007	D13	-0.022	D13	-0.001	D22	-0.00029	D21	0.00048	D8	0.00206	D8
	Min	-0.035	D8	-0.089	D8	-0.003	D9	-0.00113	D8	0.00011	D13	0.00051	D13
1165	Max	0.011	D2	-0.021	D13	0.001	D6	0.00114	D8	-0.00001	D21	0.00058	D6
	Min	0.002	D13	-0.086	D8	0.000	D13	0.00029	D21	-0.00005	D6	0.00007	D21
1166	Max	0.009	D2	-0.031	D21	0.002	D8	0.00066	D8	0.00000	D13	0.00067	D6
	Min	0.000	D12	-0.126	D8	0.000	D13	0.00017	D21	-0.00006	D8	0.00009	D21
1167	Max	0.009	D2	-0.031	D21	0.008	D8	-0.00017	D21	0.00004	D9	0.00065	D6
	Min	-0.001	D12	-0.125	D8	0.001	D13	-0.00066	D8	0.00001	D21	0.00004	D21
1168	Max	0.010	D2	-0.021	D13	0.009	D8	-0.00029	D21	0.00000	D2	0.00057	D12
	Min	0.000	D12	-0.086	D8	0.001	D13	-0.00115	D8	0.00000	D12	-0.00007	D2
1169	Max	0.002	D2	-0.005	D13	0.002	D8	-0.00020	D13	0.00016	D2	0.00040	D12
	Min	-0.005	D12	-0.019	D8	-0.001	D13	-0.00083	D8	0.00002	D13	-0.00008	D2
1170	Max	0.001	D2	-0.005	D13	0.008	D8	0.00083	D8	0.00008	D12	0.00046	D6
	Min	-0.004	D12	-0.019	D8	0.002	D13	0.00019	D13	-0.00001	D2	0.00003	D21
1171	Max	0.004	D8	0.010	D6	0.000	D22	0.00020	D12	0.00040	D2	0.00048	D9
	Min	0.001	D22	0.001	D21	-0.003	D7	-0.00041	D2	-0.00002	D12	0.00014	D22

1172	Max	0.007	D8	0.007	D6	0.004	D2	0.00014	D6	0.00004	D7	0.00038	D12
	Min	0.001	D22	-0.001	D13	-0.001	D13	0.00001	D22	-0.00003	D12	-0.00002	D2
1173	Max	0.044	D8	0.078	D8	0.001	D2	0.00017	D6	0.00003	D13	0.00326	D8
	Min	0.011	D13	0.020	D21	-0.002	D13	0.00002	D21	-0.00013	D2	0.00082	D21
1174	Max	0.042	D8	0.070	D8	0.000	D2	0.00018	D6	0.00004	D13	0.00292	D8
	Min	0.010	D13	0.018	D21	-0.002	D13	0.00002	D21	-0.00009	D2	0.00073	D21
1175	Max	0.019	D8	0.018	D8	0.000	D22	0.00024	D6	0.00007	D13	0.00076	D8
	Min	0.005	D13	0.004	D13	-0.003	D7	0.00002	D21	-0.00007	D6	0.00018	D13
1176	Max	0.031	D8	0.048	D8	0.002	D2	0.00031	D8	-0.00002	D13	0.00200	D8
	Min	0.009	D21	0.012	D13	-0.002	D13	0.00007	D21	-0.00035	D2	0.00050	D13
1177	Max	0.017	D2	0.018	D8	0.003	D2	0.00064	D8	-0.00009	D13	0.00073	D8
	Min	0.005	D13	0.004	D13	-0.001	D13	0.00019	D21	-0.00073	D2	0.00017	D13
1178	Max	0.003	D2	0.000	D1	0.000	D1	0.00026	D12	0.00000	D1	0.00080	D2
	Min	0.000	D12	0.000	D1	0.000	D1	-0.00035	D2	0.00000	D1	0.00022	D12
1179	Max	0.004	D2	0.000	D1	0.004	D2	0.00007	D12	0.00000	D1	0.00055	D6
	Min	0.001	D13	0.000	D1	0.001	D13	-0.00020	D2	0.00000	D1	0.00007	D22
1180	Max	0.009	D2	0.000	D1	0.000	D1	0.00023	D8	0.00000	D1	0.00332	D8
	Min	-0.001	D12	0.000	D1	0.000	D1	0.00004	D21	0.00000	D1	0.00083	D21
1181	Max	0.010	D2	0.000	D1	0.000	D1	0.00021	D8	0.00000	D1	0.00299	D8
	Min	0.000	D12	0.000	D1	0.000	D1	0.00004	D21	0.00000	D1	0.00075	D21
1182	Max	0.011	D2	0.000	D1	0.000	D1	0.00028	D6	0.00000	D1	0.00082	D8
	Min	0.003	D13	0.000	D1	0.000	D1	0.00002	D21	0.00000	D1	0.00020	D13
1183	Max	0.011	D2	0.000	D1	0.000	D1	0.00046	D8	0.00000	D1	0.00206	D8
	Min	0.000	D12	0.000	D1	0.000	D1	0.00012	D21	0.00000	D1	0.00051	D13
1184	Max	0.010	D2	0.000	D1	0.000	D1	0.00100	D2	0.00000	D1	0.00081	D8
	Min	0.001	D12	0.000	D1	0.000	D1	0.00026	D13	0.00000	D1	0.00019	D13
1185	Max	0.001	D2	-0.004	D21	0.005	D8	0.00035	D8	0.00029	D2	0.00053	D8
	Min	-0.004	D12	-0.016	D8	0.001	D21	0.00006	D13	0.00004	D12	0.00013	D13
1186	Max	0.005	D2	-0.004	D13	0.005	D8	0.00064	D8	-0.00002	D22	0.00082	D8
	Min	-0.001	D12	-0.018	D8	0.001	D13	0.00016	D21	-0.00013	D8	0.00019	D13
1187	Max	-0.005	D21	-0.016	D13	0.004	D8	0.00052	D8	0.00003	D8	0.00301	D8
	Min	-0.025	D8	-0.064	D8	0.001	D13	0.00012	D21	-0.00003	D13	0.00075	D21
1188	Max	-0.006	D21	-0.018	D21	0.003	D8	0.00015	D8	0.00007	D8	0.00335	D8
	Min	-0.030	D8	-0.071	D8	0.001	D13	0.00003	D21	-0.00002	D13	0.00084	D21
1189	Max	-0.002	D13	-0.011	D13	0.002	D8	-0.00010	D13	0.00007	D8	0.00210	D8
	Min	-0.015	D6	-0.045	D8	0.000	D13	-0.00042	D8	-0.00002	D13	0.00052	D13
1190	Max	0.005	D2	-0.004	D13	0.002	D8	0.00010	D2	-0.00006	D12	0.00081	D8
	Min	-0.004	D12	-0.018	D8	0.000	D13	-0.00005	D12	-0.00027	D2	0.00019	D13
1191	Max	0.002	D2	-0.003	D13	0.002	D8	-0.00009	D13	0.00004	D12	0.00050	D8
	Min	-0.005	D12	-0.016	D8	0.000	D13	-0.00036	D8	-0.00006	D9	0.00010	D21

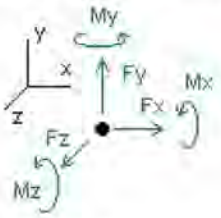
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	Min	0.000	D1	0.000	D1	0.000	D1	-0.00005	D12	0.00000	D1	-0.00238	D2
1193	Max	0.025	D2	0.012	D12	0.002	D2	0.00039	D2	0.00005	D9	0.00061	D12
	Min	-0.007	D12	-0.051	D2	0.000	D12	-0.00010	D12	-0.00002	D12	-0.00240	D2
1194	Max	0.006	D12	0.056	D2	0.001	D8	0.00015	D7	0.00009	D2	0.00050	D12
	Min	-0.028	D2	-0.012	D12	-0.002	D13	-0.00005	D12	-0.00004	D13	-0.00233	D2
1195	Max	0.000	D1	0.000	D1	0.000	D1	0.00019	D9	0.00000	D1	0.00043	D12
	Min	0.000	D1	0.000	D1	0.000	D1	-0.00010	D12	0.00000	D1	-0.00166	D2
1196	Max	0.017	D2	0.010	D12	0.001	D2	0.00007	D12	0.00001	D12	0.00048	D12
	Min	-0.005	D12	-0.036	D2	-0.001	D12	-0.00041	D2	-0.00016	D2	-0.00169	D2
1197	Max	0.005	D12	0.039	D2	0.002	D8	0.00014	D9	0.00019	D2	0.00037	D12
	Min	-0.019	D2	-0.009	D12	-0.002	D13	-0.00011	D12	-0.00003	D13	-0.00161	D2
1198	Max	0.010	D2	0.000	D1	0.000	D1	0.00030	D8	0.00000	D1	0.00298	D8
	Min	-0.001	D12	0.000	D1	0.000	D1	0.00006	D21	0.00000	D1	0.00075	D21
1199	Max	0.041	D8	0.070	D8	0.001	D2	0.00020	D8	0.00001	D13	0.00292	D8
	Min	0.012	D13	0.018	D21	-0.002	D13	0.00003	D21	-0.00020	D2	0.00073	D21
1200	Max	-0.005	D21	-0.016	D21	0.003	D8	-0.00005	D13	0.00010	D8	0.00301	D8
	Min	-0.025	D8	-0.064	D8	0.000	D13	-0.00020	D8	-0.00001	D13	0.00075	D21
1201	Max	0.012	D2	0.000	D1	0.000	D1	0.00024	D8	0.00000	D1	0.00206	D8
	Min	0.002	D13	0.000	D1	0.000	D1	0.00004	D21	0.00000	D1	0.00051	D13
1202	Max	0.034	D8	0.048	D8	0.000	D22	0.00021	D6	0.00004	D13	0.00200	D8
	Min	0.008	D13	0.012	D13	-0.002	D7	0.00003	D21	-0.00008	D8	0.00050	D13
1203	Max	-0.002	D21	-0.011	D13	0.004	D8	0.00063	D8	-0.00002	D22	0.00210	D8
	Min	-0.012	D6	-0.045	D8	0.001	D13	0.00016	D21	-0.00007	D9	0.00052	D13
1204	Max	0.000	D2	0.000	D12	0.000	D9	0.00023	D9	0.00001	D9	0.00016	D12
	Min	0.000	D12	-0.001	D9	0.000	D12	0.00004	D12	-0.00001	D12	-0.00023	D2
1205	Max	0.000	D13	0.000	D12	0.002	D2	-0.00004	D13	0.00002	D6	0.00036	D6
	Min	0.000	D8	-0.001	D2	0.000	D12	-0.00064	D2	0.00000	D13	-0.00006	D13
1206	Max	0.003	D2	0.001	D12	0.000	D9	0.00014	D9	0.00000	D13	0.00065	D12
	Min	-0.001	D12	-0.005	D2	0.000	D12	-0.00006	D12	-0.00001	D2	-0.00269	D2
1207	Max	0.002	D2	0.001	D12	0.000	D9	0.00022	D9	0.00001	D9	0.00038	D12
	Min	0.000	D12	-0.003	D2	0.000	D12	-0.00004	D12	-0.00001	D12	-0.00161	D2
1208	Max	0.000	D2	0.000	D12	0.000	D9	0.00023	D9	0.00001	D13	0.00013	D12
	Min	0.000	D12	-0.001	D2	0.000	D12	-0.00002	D12	-0.00001	D8	-0.00058	D2
1209	Max	0.002	D2	0.001	D12	0.000	D9	0.00012	D9	0.00000	D12	0.00060	D12
	Min	-0.001	D12	-0.005	D2	0.000	D12	-0.00008	D12	-0.00003	D2	-0.00242	D2
1210	Max	0.000	D2	0.000	D12	0.001	D2	0.00040	D2	0.00000	D12	0.00020	D12
	Min	0.000	D12	-0.001	D2	0.000	D12	-0.00011	D12	-0.00003	D2	-0.00064	D2
1211	Max	0.002	D2	0.001	D12	0.000	D9	0.00018	D9	0.00001	D13	0.00057	D12
	Min	-0.001	D12	-0.005	D2	0.000	D12	-0.00005	D12	0.00000	D6	-0.00239	D2
1212	Max	0.002	D2	0.001	D12	0.000	D2	0.00014	D9	0.00001	D12	0.00044	D12

	Min	0.000	D12	-0.003	D2	0.000	D12	-0.00009	D12	-0.00004	D2	-0.00167	D2
1214	Max	0.000	D2	0.000	D13	0.001	D2	0.00059	D2	0.00002	D2	0.00025	D12
	Min	0.000	D12	-0.001	D2	0.000	D13	-0.00003	D13	-0.00001	D12	-0.00012	D2
1215	Max	0.000	D2	0.000	D12	0.000	D2	0.00058	D2	0.00001	D2	0.00022	D12
	Min	0.000	D12	-0.001	D2	0.000	D12	-0.00017	D12	-0.00001	D12	-0.00023	D2
1216	Max	0.000	D2	0.000	D12	0.000	D2	0.00051	D2	0.00002	D2	0.00023	D12
	Min	0.000	D12	-0.001	D2	0.000	D12	-0.00013	D12	-0.00001	D12	-0.00017	D2
1217	Max	0.000	D2	0.000	D13	0.000	D2	0.00054	D2	0.00002	D2	0.00024	D12
	Min	0.000	D12	-0.001	D2	0.000	D13	0.00000	D13	-0.00001	D12	-0.00014	D2
1218	Max	0.000	D2	0.000	D13	0.000	D2	0.00055	D2	0.00001	D2	0.00027	D12
	Min	0.000	D12	-0.001	D2	0.000	D13	-0.00001	D13	-0.00001	D12	-0.00013	D2
1219	Max	0.000	D2	0.000	D13	0.001	D8	0.00063	D8	0.00001	D2	0.00030	D12
	Min	0.000	D12	-0.001	D8	0.000	D13	0.00007	D13	-0.00001	D12	-0.00021	D2
1220	Max	0.001	D2	0.000	D21	0.001	D8	0.00085	D8	0.00003	D2	0.00037	D12
	Min	0.000	D12	-0.002	D8	0.000	D21	0.00020	D21	0.00000	D12	-0.00047	D2
1221	Max	0.002	D2	0.000	D21	0.000	D12	0.00025	D12	0.00000	D12	0.00075	D2
	Min	-0.001	D12	-0.001	D8	0.000	D2	-0.00027	D2	-0.00002	D9	0.00022	D12
1223	Max	0.000	D2	0.000	D12	0.009	D2	-0.00014	D12	0.00001	D12	0.00041	D12
	Min	0.000	D12	-0.001	D2	-0.001	D13	-0.00059	D2	-0.00002	D2	-0.00015	D2
1224	Max	0.000	D9	0.001	D12	0.009	D2	0.00025	D12	0.00001	D12	0.00040	D12
	Min	0.000	D12	-0.001	D2	-0.002	D12	-0.00067	D2	-0.00002	D2	-0.00011	D9
1225	Max	0.000	D2	0.000	D12	0.010	D2	0.00017	D12	0.00001	D12	0.00040	D12
	Min	0.000	D12	-0.001	D2	-0.004	D12	-0.00050	D2	-0.00002	D2	-0.00013	D2
1226	Max	0.000	D2	0.000	D12	0.010	D2	0.00000	D12	0.00001	D12	0.00040	D12
	Min	0.000	D12	-0.001	D2	-0.001	D12	-0.00054	D2	-0.00002	D2	-0.00014	D2
1227	Max	0.000	D2	0.000	D21	0.010	D8	-0.00016	D21	0.00001	D12	0.00040	D12
	Min	0.000	D12	-0.001	D2	-0.001	D13	-0.00054	D2	-0.00002	D2	-0.00021	D2
1228	Max	0.000	D2	0.000	D21	0.013	D8	-0.00015	D21	0.00001	D12	0.00036	D12
	Min	0.000	D12	-0.001	D8	0.001	D13	-0.00062	D8	-0.00002	D2	-0.00038	D2
1229	Max	0.001	D2	0.000	D13	0.010	D8	-0.00019	D13	0.00000	D12	0.00023	D12
	Min	0.000	D12	-0.002	D8	0.001	D13	-0.00085	D8	-0.00003	D2	-0.00091	D2
1230	Max	0.004	D2	0.000	D13	0.004	D2	0.00003	D12	0.00002	D12	0.00053	D6
	Min	0.000	D12	-0.001	D8	0.001	D13	-0.00022	D2	0.00000	D7	0.00007	D22
1234	Max	0.006	D2	-0.002	D21	0.000	D8	0.00023	D8	0.00002	D8	0.00333	D8
	Min	-0.003	D12	-0.007	D8	0.000	D21	0.00004	D21	0.00000	D13	0.00083	D21
1235	Max	0.008	D2	-0.002	D13	0.000	D8	0.00024	D8	0.00000	D2	0.00300	D8
	Min	-0.002	D12	-0.006	D8	0.000	D21	0.00005	D21	-0.00001	D13	0.00075	D21
1236	Max	0.010	D2	0.000	D13	0.000	D6	0.00031	D6	0.00000	D8	0.00083	D8
	Min	0.003	D13	-0.002	D8	0.000	D21	0.00003	D21	-0.00001	D13	0.00020	D13
1237	Max	0.010	D2	-0.001	D13	0.001	D8	0.00040	D8	0.00005	D8	0.00207	D8
	Min	-0.001	D12	-0.004	D8	0.000	D21	0.00011	D21	0.00001	D13	0.00051	D13

1238	Max	0.009	D2	0.000	D13	0.001	D2	0.00094	D2	0.00003	D8	0.00082	D8
	Min	0.000	D12	-0.002	D8	0.000	D13	0.00024	D13	0.00000	D13	0.00019	D13
1239	Max	0.008	D2	-0.002	D21	0.000	D8	0.00026	D8	0.00004	D8	0.00300	D8
	Min	-0.003	D12	-0.006	D8	0.000	D21	0.00006	D21	0.00000	D13	0.00075	D21
1240	Max	0.010	D2	-0.001	D13	0.000	D8	0.00028	D8	0.00000	D15	0.00208	D8
	Min	0.001	D13	-0.004	D8	0.000	D21	0.00005	D21	-0.00001	D13	0.00051	D13

Envelope for nodal reactions

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



Direction of positive forces and moments

Envelope of nodal reactions for :

D1=DL
D2=DL+LL
D3=DL+SL
D4=DL+0.75LL
D5=DL+0.75SL
D6=DL+0.6Wx
D7=DL+0.6Wz
D8=DL+0.45Wx+0.75LL
D9=DL+0.45Wz+0.75LL
D10=DL+0.45Wx+0.75SL
D11=DL+0.45Wz+0.75SL
D12=0.6DL+0.6Wx
D13=0.6DL+0.6Wz
D14=DL+0.7EQx
D15=DL+0.7EQz
D16=DL+0.525EQx
D17=DL+0.525EQz
D18=DL+0.75SL
D19=DL+0.525EQx+0.75SL
D20=DL+0.525EQz+0.75SL
D21=0.6DL+0.7EQx
D22=0.6DL+0.7EQz
S1=DL
S2=DL+LL
S3=DL+SL
S4=DL+0.75LL
S5=DL+0.75SL
S6=DL+0.6Wx
S7=DL+0.6Wz
S8=DL+0.7EQx
S9=DL+0.7EQz
S10=DL+0.45Wx+0.75LL
S11=DL+0.45Wz+0.75LL

S12=DL+0.45Wx+0.75SL
S13=DL+0.45Wz+0.75SL
S14=0.6DL+0.6Wx
S15=0.6DL+0.6Wz
S16=DL+0.7EQx
S17=DL+0.7EQz
S18=DL+0.525EQx
S19=DL+0.525EQz
S20=DL+0.75SL
S21=DL+0.525EQx+0.75SL
S22=DL+0.525EQz+0.75SL
S23=0.6DL+0.7EQx
S24=0.6DL+0.7EQz

		Forces						Moments					
Node		**Fx** [Kip]	**lc**	**Fy** [Kip]	**lc**	**Fz** [Kip]	**lc**	**Mx** [Kip*ft]	**lc**	**My** [Kip*ft]	**lc**	**Mz** [Kip*ft]	**lc**
1016	Max	0.738	D12	0.151	D12	0.705	D12	0.00000	D1	-0.00027	D12	0.00000	D1
	Min	-1.732	D2	-0.512	D7	-1.725	D2	0.00000	D1	-0.00233	D2	0.00000	D1
1017	Max	0.784	D2	0.596	D2	0.000	D1	0.00000	D1	0.01187	D2	0.00000	D1
	Min	0.031	D12	0.030	D12	0.000	D1	0.00000	D1	-0.00416	D12	0.00000	D1
1018	Max	0.024	D12	1.671	D2	1.161	D7	0.00000	D1	-0.00746	D22	0.00000	D1
	Min	-0.070	D2	-0.317	D13	-0.013	D22	0.00000	D1	-0.02451	D2	0.00000	D1
1019	Max	1.307	D12	1.166	D2	0.004	D13	0.00000	D1	0.01611	D2	0.00000	D1
	Min	-0.142	D9	0.065	D12	-0.056	D2	0.00000	D1	-0.00320	D12	0.00000	D1
1020	Max	0.000	D12	1.701	D9	0.000	D1	0.00000	D1	0.02648	D2	0.00000	D1
	Min	-0.098	D2	0.405	D12	0.000	D1	0.00000	D1	-0.00412	D12	0.00000	D1
1024	Max	0.039	D12	1.499	D2	2.380	D9	0.00000	D1	-0.00151	D22	0.00000	D1
	Min	-0.057	D2	-0.141	D12	-0.649	D12	0.00000	D1	-0.00497	D2	0.00000	D1
1025	Max	0.212	D2	1.279	D2	0.000	D1	0.00000	D1	0.01438	D2	0.00000	D1
	Min	0.013	D13	-0.210	D12	0.000	D1	0.00000	D1	-0.00410	D12	0.00000	D1
1026	Max	0.536	D2	1.713	D2	0.010	D13	0.00000	D1	0.00036	D12	0.00000	D1
	Min	0.035	D13	-0.301	D12	-0.029	D2	0.00000	D1	-0.00333	D9	0.00000	D1
1028	Max	2.881	D2	2.593	D2	0.014	D13	0.00000	D1	0.00014	D12	0.00000	D1
	Min	-0.200	D12	-0.465	D12	-0.043	D2	0.00000	D1	-0.00229	D2	0.00000	D1
1032	Max	1.344	D6	1.339	D2	0.004	D12	0.00000	D1	0.02864	D2	0.00000	D1
	Min	0.008	D21	0.103	D12	-0.120	D2	0.00000	D1	-0.00593	D12	0.00000	D1
1034	Max	1.637	D8	2.410	D2	0.007	D12	0.00000	D1	0.03677	D2	0.00000	D1
	Min	0.372	D21	-0.249	D12	-0.822	D2	0.00000	D1	-0.00882	D12	0.00000	D1
1037	Max	0.028	D12	1.789	D2	0.924	D9	0.00000	D1	-0.00350	D22	0.00000	D1
	Min	-0.044	D2	-0.221	D13	-0.441	D12	0.00000	D1	-0.01151	D2	0.00000	D1
1038	Max	0.040	D8	1.566	D9	0.000	D1	0.00000	D1	0.01837	D2	0.00000	D1
	Min	-0.004	D13	-0.144	D12	0.000	D1	0.00000	D1	-0.00409	D12	0.00000	D1
1047	Max	0.024	D12	1.771	D2	0.971	D7	0.00000	D1	-0.00547	D22	0.00000	D1
	Min	-0.046	D2	-0.293	D13	-0.127	D12	0.00000	D1	-0.01799	D2	0.00000	D1
1048	Max	0.013	D12	1.724	D9	0.000	D1	0.00000	D1	0.02237	D2	0.00000	D1
	Min	-0.033	D2	0.152	D12	0.000	D1	0.00000	D1	-0.00408	D12	0.00000	D1
1115	Max	0.030	D12	1.768	D2	1.104	D7	0.00000	D1	-0.00948	D22	0.00000	D1
	Min	-0.142	D2	-0.267	D13	0.026	D22	0.00000	D1	-0.03112	D2	0.00000	D1

1116	Max	-0.035	D12	1.707	D9	0.000	D1	0.00000	D1	0.03070	D2	0.00000	D1
	Min	-0.234	D2	0.488	D21	0.000	D1	0.00000	D1	-0.00418	D12	0.00000	D1
1133	Max	0.053	D12	1.934	D8	0.938	D9	0.00000	D1	-0.01154	D22	0.00000	D1
	Min	-0.352	D2	-0.208	D13	0.122	D22	0.00000	D1	-0.03790	D2	0.00000	D1
1134	Max	-0.133	D12	1.668	D8	0.000	D1	0.00000	D1	0.03511	D2	0.00000	D1
	Min	-0.616	D2	0.446	D21	0.000	D1	0.00000	D1	-0.00427	D12	0.00000	D1
1154	Max	0.122	D12	1.678	D8	1.166	D8	0.00000	D1	-0.01379	D22	0.00000	D1
	Min	-0.993	D2	-0.122	D13	0.242	D22	0.00000	D1	-0.04526	D2	0.00000	D1
1155	Max	-0.381	D13	1.471	D8	0.000	D1	0.00000	D1	0.04013	D2	0.00000	D1
	Min	-1.779	D2	0.381	D21	0.000	D1	0.00000	D1	-0.00425	D12	0.00000	D1
1178	Max	0.000	D1	0.636	D2	0.933	D2	0.00000	D1	-0.00191	D13	0.00000	D1
	Min	0.000	D1	-0.400	D12	-0.088	D12	0.00000	D1	-0.00915	D8	0.00000	D1
1179	Max	0.000	D1	1.027	D2	0.000	D1	0.00000	D1	0.00119	D7	0.00000	D1
	Min	0.000	D1	-0.002	D12	0.000	D1	0.00000	D1	-0.01086	D12	0.00000	D1
1180	Max	0.000	D1	1.225	D2	-0.006	D13	0.00000	D1	-0.00270	D13	0.00000	D1
	Min	0.000	D1	0.265	D13	-0.094	D2	0.00000	D1	-0.02201	D8	0.00000	D1
1181	Max	0.000	D1	1.311	D2	0.004	D13	0.00000	D1	-0.00016	D13	0.00000	D1
	Min	0.000	D1	0.221	D13	-0.055	D8	0.00000	D1	-0.00918	D8	0.00000	D1
1182	Max	0.000	D1	2.105	D2	0.305	D2	0.00000	D1	-0.00028	D13	0.00000	D1
	Min	0.000	D1	0.587	D13	-0.012	D12	0.00000	D1	-0.00344	D8	0.00000	D1
1183	Max	0.000	D1	1.378	D2	-0.109	D13	0.00000	D1	-0.00634	D13	0.00000	D1
	Min	0.000	D1	0.393	D22	-0.610	D2	0.00000	D1	-0.04186	D8	0.00000	D1
1184	Max	0.000	D1	2.301	D8	-0.341	D13	0.00000	D1	-0.00529	D13	0.00000	D1
	Min	0.000	D1	0.607	D13	-1.791	D2	0.00000	D1	-0.04073	D8	0.00000	D1
1192	Max	1.019	D6	1.356	D2	0.008	D13	0.00000	D1	0.00392	D2	0.00000	D1
	Min	-0.107	D13	-0.005	D12	-0.032	D2	0.00000	D1	-0.00086	D12	0.00000	D1
1195	Max	1.116	D6	1.608	D2	0.006	D12	0.00000	D1	0.03625	D2	0.00000	D1
	Min	0.108	D21	0.011	D12	-0.296	D2	0.00000	D1	-0.00792	D12	0.00000	D1
1198	Max	0.000	D1	1.299	D2	-0.035	D13	0.00000	D1	-0.00527	D13	0.00000	D1
	Min	0.000	D1	0.361	D13	-0.229	D2	0.00000	D1	-0.03482	D8	0.00000	D1
1201	Max	0.000	D1	1.442	D2	0.077	D9	0.00000	D1	0.00083	D13	0.00000	D1
	Min	0.000	D1	0.287	D13	-0.030	D12	0.00000	D1	-0.00248	D6	0.00000	D1

Current Date: 9/15/2026 5:49 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+SL
D4=DL+0.75LL
D5=DL+0.75SL
D6=DL+0.6Wx
D7=DL+0.6Wz
D8=DL+0.45Wx+0.75LL
D9=DL+0.45Wz+0.75LL
D10=DL+0.45Wx+0.75SL
D11=DL+0.45Wz+0.75SL
D12=0.6DL+0.6Wx
D13=0.6DL+0.6Wz
D14=DL+0.7EQx
D15=DL+0.7EQz
D16=DL+0.525EQx
D17=DL+0.525EQz
D18=DL+0.75SL
D19=DL+0.525EQx+0.75SL
D20=DL+0.525EQz+0.75SL
D21=0.6DL+0.7EQx
D22=0.6DL+0.7EQz

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BOTTOM BRACING	***aisiSSMA_S 1000S250-97***	**1668**	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1669	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1670	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1671	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1672	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1673	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1674	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1675	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1676	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1677	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1678	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1679	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1680	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1681	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1682	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1683	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1684	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1685	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1686	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1687	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1688	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1689	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1690	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1691	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1692	D2 at 50.00%	0.02	OK	Eq. H1.1-1

		1693	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1694	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1695	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1696	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1697	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1698	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1699	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1700	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1701	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1702	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1703	D2 at 50.00%	0.02	OK	Eq. H1.1-1
		1704	D2 at 50.00%	0.01	OK	Eq. H1.1-1
		1705	D2 at 50.00%	0.01	OK	Eq. H1.1-1
		1706	D2 at 50.00%	0.01	OK	Eq. H1.1-1
		1707	D2 at 50.00%	0.01	OK	Eq. H1.1-1
		1708	D2 at 50.00%	0.01	OK	Eq. H1.1-1
		1709	D2 at 50.00%	0.01	OK	Eq. H1.1-1
BOTTOM CHORD		**1399**	D13 at 12.50%	0.36	OK	Eq. H1.2-1
		1400	D2 at 12.50%	0.41	OK	Eq. H1.2-1
		1403	D2 at 100.00%	0.24	OK	Sec. G5
		1405	D2 at 100.00%	0.36	OK	Sec. G5
		1406	D2 at 100.00%	**0.54**	**OK**	Sec. G5
		1407	D2 at 100.00%	0.28	OK	Sec. G5
		1408	D2 at 100.00%	0.50	OK	Sec. G5
		1411	D7 at 100.00%	0.11	OK	Sec. G5
		1420	D12 at 50.00%	0.50	OK	Eq. H1.2-1
		1424	D2 at 0.00%	0.17	OK	Sec. G5
		1425	D2 at 0.00%	0.37	OK	Sec. G5
		1542	D2 at 0.00%	0.37	OK	Sec. G5
		1561	D8 at 0.00%	0.40	OK	Sec. G5
		1580	D2 at 87.50%	0.43	OK	Eq. H1.2-1
		1605	D8 at 100.00%	0.48	OK	Sec. G5
		1606	D2 at 100.00%	0.21	OK	Sec. G5
		1607	D2 at 100.00%	0.29	OK	Sec. G5
		1608	D2 at 100.00%	0.25	OK	Sec. G5
		1609	D2 at 100.00%	0.27	OK	Sec. G5
		1610	D2 at 100.00%	0.44	OK	Sec. G5
		1611	D2 at 100.00%	0.13	OK	Sec. G5
		1710	D2 at 100.00%	0.28	OK	Sec. G5
		1713	D2 at 100.00%	0.33	OK	Sec. G5
		1715	D2 at 100.00%	0.27	OK	Sec. G5
		1717	D2 at 100.00%	0.30	OK	Sec. G5
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BRACE	***aisiSSMA_S 400S162-43***	**1429**	D2 at 100.00%	0.04	OK	Sec. E3
		1430	D2 at 100.00%	0.06	OK	Sec. E3
		1431	D2 at 100.00%	0.04	OK	Sec. E3
		1432	D9 at 0.00%	0.03	OK	Sec. D2
		1433	D2 at 0.00%	0.03	OK	Sec. D2
		1434	D2 at 0.00%	0.04	OK	Sec. D2
		1435	D9 at 0.00%	0.04	OK	Sec. D2
		1436	D9 at 0.00%	0.03	OK	Sec. D2
		1437	D2 at 0.00%	0.02	OK	Sec. D2
		1438	D2 at 100.00%	0.11	OK	Sec. E3
		1439	D2 at 100.00%	0.06	OK	Sec. E3
		1440	D2 at 100.00%	0.06	OK	Sec. E3
		1441	D2 at 100.00%	0.10	OK	Sec. E3
		1442	D2 at 100.00%	0.08	OK	Sec. E3
		1443	D2 at 100.00%	0.07	OK	Sec. E3
		1444	D2 at 100.00%	0.02	OK	Sec. E3
		1445	D2 at 0.00%	0.02	OK	Sec. D2
		1446	D2 at 0.00%	0.03	OK	Sec. D2
		1447	D9 at 0.00%	0.04	OK	Sec. D2
		1448	D9 at 0.00%	0.02	OK	Sec. D2
		1449	D9 at 100.00%	0.02	OK	Sec. E3

1450	D2 at 100.00%	0.02	OK	Sec. E3
1451	D8 at 0.00%	0.01	OK	Sec. D2
1452	D9 at 0.00%	0.02	OK	Sec. D2
1453	D2 at 100.00%	0.02	OK	Sec. E3
1454	D2 at 100.00%	0.03	OK	Sec. E3
1455	D2 at 100.00%	0.03	OK	Sec. E3
1456	D2 at 50.00%	0.03	OK	Eq. H1.1-1
1457	D2 at 50.00%	0.09	OK	Eq. H1.1-1
1458	D2 at 50.00%	0.12	OK	Eq. H1.1-1
1459	D2 at 50.00%	0.03	OK	Eq. H1.1-1
1460	D2 at 50.00%	0.02	OK	Eq. H1.1-1
1461	D2 at 50.00%	0.06	OK	Eq. H1.1-1
1462	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1463	D12 at 43.75%	0.02	OK	Eq. H1.2-1
1464	D2 at 50.00%	0.03	OK	Eq. H1.1-1
1465	D2 at 50.00%	0.09	OK	Eq. H1.1-1
1466	D2 at 50.00%	0.13	OK	Eq. H1.1-1
1467	D2 at 50.00%	0.01	OK	Eq. H1.1-1
1468	D12 at 43.75%	0.04	OK	Eq. H1.2-1
1469	D12 at 43.75%	0.08	OK	Eq. H1.2-1
1470	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1471	D12 at 43.75%	0.01	OK	Eq. H1.2-1
1472	D13 at 43.75%	0.05	OK	Eq. H1.2-1
1473	D9 at 43.75%	0.14	OK	Eq. H1.2-1
1474	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1475	D2 at 50.00%	0.01	OK	Eq. H1.1-1
1476	D9 at 50.00%	0.02	OK	Eq. H1.1-1
1477	D2 at 43.75%	0.11	OK	Eq. H1.2-1
1478	D8 at 50.00%	0.04	OK	Eq. H1.2-1
1479	D2 at 50.00%	0.01	OK	Eq. H1.1-1
1480	D2 at 43.75%	0.12	OK	Eq. H1.2-1
1481	D2 at 43.75%	0.15	OK	Eq. H1.2-1
1482	D2 at 50.00%	0.04	OK	Eq. H1.2-1
1483	D2 at 50.00%	0.01	OK	Eq. H1.1-1
1484	D9 at 43.75%	**0.18**	**OK**	Eq. H1.2-1
1485	D9 at 43.75%	0.17	OK	Eq. H1.2-1
1486	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1487	D2 at 50.00%	0.01	OK	Eq. H1.1-1
1546	D8 at 100.00%	0.06	OK	Sec. E3
1547	D2 at 0.00%	0.04	OK	Sec. D2
1548	D9 at 0.00%	0.04	OK	Sec. D2
1549	D8 at 100.00%	0.02	OK	Sec. E3
1550	D2 at 0.00%	0.01	OK	Sec. D2
1551	D9 at 0.00%	0.02	OK	Sec. D2
1552	D8 at 100.00%	0.02	OK	Sec. E3
1553	D13 at 43.75%	0.06	OK	Eq. H1.2-1
1554	D9 at 43.75%	0.14	OK	Eq. H1.2-1
1555	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1556	D8 at 50.00%	0.01	OK	Eq. H1.1-1
1557	D9 at 50.00%	0.03	OK	Eq. H1.1-1
1558	D2 at 43.75%	0.11	OK	Eq. H1.2-1
1559	D2 at 50.00%	0.04	OK	Eq. H1.2-1
1560	D8 at 50.00%	0.02	OK	Eq. H1.1-1
1565	D8 at 100.00%	0.07	OK	Sec. E3
1566	D2 at 0.00%	0.03	OK	Sec. D2
1567	D9 at 0.00%	0.03	OK	Sec. D2
1568	D8 at 100.00%	0.09	OK	Sec. E3
1569	D8 at 100.00%	0.08	OK	Sec. E3
1570	D8 at 100.00%	0.03	OK	Sec. E3
1571	D8 at 100.00%	0.03	OK	Sec. E3
1572	D8 at 50.00%	0.02	OK	Eq. H1.1-1
1573	D8 at 50.00%	0.06	OK	Eq. H1.1-1
1574	D7 at 50.00%	0.03	OK	Eq. H1.2-1
1575	D8 at 50.00%	0.02	OK	Eq. H1.1-1

		1576	D8 at 50.00%	0.02	OK	Eq. H1.1-1
		1577	D8 at 50.00%	0.07	OK	Eq. H1.1-1
		1578	D2 at 50.00%	0.01	OK	Eq. H1.2-1
		1579	D8 at 50.00%	0.02	OK	Eq. H1.1-1
		1584	D8 at 100.00%	0.05	OK	Sec. E3
		1585	D9 at 0.00%	0.02	OK	Sec. D2
		1586	D2 at 0.00%	0.02	OK	Sec. D2
		1587	D8 at 100.00%	0.12	OK	Sec. E3
		1588	D8 at 100.00%	0.07	OK	Sec. E3
		1589	D8 at 100.00%	0.07	OK	Sec. E3
		1590	D8 at 100.00%	0.12	OK	Sec. E3
		1591	D8 at 50.00%	0.03	OK	Eq. H1.1-1
		1592	D8 at 50.00%	0.10	OK	Eq. H1.1-1
		1593	D8 at 50.00%	0.14	OK	Eq. H1.1-1
		1594	D8 at 50.00%	0.04	OK	Eq. H1.1-1
		1595	D8 at 50.00%	0.03	OK	Eq. H1.1-1
		1596	D8 at 50.00%	0.10	OK	Eq. H1.1-1
		1597	D8 at 50.00%	0.14	OK	Eq. H1.1-1
		1598	D8 at 50.00%	0.03	OK	Eq. H1.1-1
EAVE MEMBER	***aisiSSMA_T 1000T200-54***	**1412**	D7 at 50.00%	**0.38**	**With warnings**	Eq. H1.2-1
		1413	D6 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1414	D6 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1415	D6 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1416	D6 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1417	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1418	D6 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1419	D7 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1512	D7 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1513	D7 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1514	D7 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1531	D7 at 50.00%	0.16	With warnings	Eq. H1.2-1
		1532	D7 at 50.00%	0.16	With warnings	Eq. H1.2-1
		1533	D7 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1599	D6 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1600	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1601	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1602	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1603	D6 at 50.00%	0.05	With warnings	Eq. H1.1-1
		1604	D6 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1619	D7 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1620	D7 at 50.00%	0.05	With warnings	Eq. H1.1-1
		1621	D7 at 50.00%	0.05	With warnings	Eq. H1.1-1
		1622	D7 at 50.00%	0.05	With warnings	Eq. H1.1-1
		1623	D7 at 50.00%	0.11	With warnings	Eq. H1.2-1
		1624	D7 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1625	D7 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1626	D7 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1712	D6 at 50.00%	0.04	With warnings	Eq. H1.2-1
		1753	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
		1754	D6 at 50.00%	0.05	With warnings	Eq. H1.1-1
		1755	D6 at 50.00%	0.04	With warnings	Eq. H1.1-1
PEAK TRUSS	***aisiSSMA_S 400S162-43***	**1398**	D2 at 100.00%	**0.07**	**OK**	Sec. D2
PERIMETER MEMBER	***aisiSSMA_S 1000S250-97***	**1488**	D2 at 56.25%	0.08	OK	Eq. H1.2-1
		1489	D2 at 43.75%	0.06	OK	Eq. H1.2-1
		1490	D2 at 43.75%	0.06	OK	Eq. H1.2-1
		1491	D2 at 0.00%	0.06	OK	Sec. G5
		1492	D2 at 0.00%	0.06	OK	Sec. G5
		1493	D2 at 0.00%	0.06	OK	Sec. G5
		1494	D2 at 43.75%	0.06	OK	Eq. H1.2-1
		1495	D2 at 43.75%	0.07	OK	Eq. H1.2-1
		1496	D2 at 37.50%	0.04	OK	Eq. H1.2-1

1502	D9 at 50.00%	0.03	OK	Eq. H1.2-1
1503	D9 at 50.00%	0.03	OK	Eq. H1.2-1
1504	D9 at 50.00%	0.03	OK	Eq. H1.2-1
1505	D9 at 50.00%	0.02	OK	Eq. H1.2-1
1506	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1507	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1508	D9 at 50.00%	0.05	OK	Eq. H1.2-1
1509	D9 at 50.00%	0.07	OK	Eq. H1.2-1
1510	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1515	D2 at 0.00%	0.01	OK	Sec. E4
1516	D8 at 50.00%	0.02	OK	Eq. H1.2-1
1517	D8 at 50.00%	0.02	OK	Eq. H1.2-1
1518	D8 at 50.00%	0.02	OK	Eq. H1.2-1
1519	D8 at 50.00%	0.02	OK	Eq. H1.2-1
1520	D2 at 0.00%	0.02	OK	Sec. E4
1521	D9 at 50.00%	0.01	OK	Eq. H1.2-1
1522	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1523	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1524	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1525	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1526	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1527	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1528	D9 at 50.00%	0.07	OK	Eq. H1.2-1
1529	D9 at 50.00%	0.08	OK	Eq. H1.2-1
1530	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1535	D2 at 37.50%	0.07	OK	Eq. H1.2-1
1536	D2 at 62.50%	0.06	OK	Eq. H1.2-1
1537	D2 at 100.00%	0.04	OK	Sec. G5
1539	D2 at 0.00%	0.04	OK	Sec. G5
1540	D2 at 37.50%	0.05	OK	Eq. H1.2-1
1541	D2 at 0.00%	0.04	OK	Sec. G5
1627	D7 at 50.00%	0.03	OK	Eq. H1.2-1
1628	D7 at 50.00%	0.03	OK	Eq. H1.2-1
1629	D7 at 50.00%	0.03	OK	Eq. H1.2-1
1630	D9 at 50.00%	0.02	OK	Eq. H1.2-1
1631	D8 at 50.00%	0.01	OK	Eq. H1.2-1
1632	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1633	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1634	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1635	D6 at 50.00%	0.03	OK	Eq. H1.2-1
1636	D8 at 50.00%	0.02	OK	Eq. H1.2-1
1637	D8 at 0.00%	0.02	OK	Sec. E4
1638	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1639	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1640	D7 at 50.00%	0.02	OK	Eq. H1.2-1
1641	D2 at 37.50%	0.10	OK	Eq. H1.2-1
1642	D2 at 37.50%	0.06	OK	Eq. H1.2-1
1643	D2 at 100.00%	0.04	OK	Sec. G5
1644	D2 at 37.50%	0.07	OK	Eq. H1.2-1
1645	D2 at 37.50%	0.06	OK	Eq. H1.2-1
1646	D2 at 0.00%	0.04	OK	Sec. G5
1647	D2 at 43.75%	0.06	OK	Eq. H1.2-1
1648	D8 at 0.00%	0.07	OK	Sec. G5
1649	D8 at 0.00%	0.07	OK	Sec. G5
1650	D8 at 0.00%	0.05	OK	Sec. G5
1651	D8 at 0.00%	0.07	OK	Sec. G5
1652	D2 at 37.50%	0.05	OK	Eq. H1.2-1
1653	D8 at 0.00%	0.07	OK	Sec. G5
1654	D8 at 0.00%	0.05	OK	Sec. G5
1655	D2 at 43.75%	0.06	OK	Eq. H1.2-1
1656	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1657	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1658	D9 at 50.00%	0.06	OK	Eq. H1.2-1
1659	D9 at 50.00%	0.07	OK	Eq. H1.2-1

	1660	D9 at 50.00%	0.08	OK	Eq. H1.2-1
	1661	D9 at 50.00%	0.07	OK	Eq. H1.2-1
	1662	D9 at 50.00%	0.05	OK	Eq. H1.2-1
	1663	D9 at 50.00%	0.05	OK	Eq. H1.2-1
	1664	D9 at 50.00%	0.05	OK	Eq. H1.2-1
	1665	D9 at 50.00%	0.06	OK	Eq. H1.2-1
	1666	D9 at 50.00%	0.07	OK	Eq. H1.2-1
	1511	D9 at 50.00%	0.06	OK	Eq. H1.2-1
	1667	D9 at 50.00%	0.06	OK	Eq. H1.2-1
	1719	D8 at 50.00%	0.02	OK	Eq. H1.2-1
	1720	D8 at 100.00%	0.07	OK	Eq. H1.2-1
	1756	D6 at 50.00%	0.03	OK	Eq. H1.2-1
	1757	D8 at 50.00%	0.02	OK	Eq. H1.2-1
PERIMETER MEMBER ...	**1758**	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1759	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1760	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1761	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1762	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1763	D12 at 50.00%	0.02	OK	Eq. H1.2-1
	1764	D12 at 50.00%	0.02	OK	Eq. H1.2-1
	1765	D8 at 50.00%	0.02	OK	Eq. H1.1-1
	1766	D9 at 50.00%	0.03	OK	Eq. H1.1-1
	1767	D7 at 50.00%	0.02	OK	Eq. H1.1-1
	1768	D7 at 50.00%	0.02	OK	Eq. H1.1-1
	1769	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1770	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1771	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1772	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1773	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1774	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1775	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1776	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1777	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1778	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1779	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1780	D7 at 50.00%	0.02	OK	Eq. H1.2-1
	1781	D2 at 50.00%	0.04	OK	Eq. H1.1-1
	1782	D2 at 50.00%	0.04	OK	Eq. H1.1-1
	1783	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1784	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1785	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1786	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1787	D6 at 50.00%	0.02	OK	Eq. H1.2-1
	1788	D12 at 50.00%	0.02	OK	Eq. H1.2-1
	1789	D12 at 50.00%	0.02	OK	Eq. H1.2-1
TOP CHORD	**1401**	D2 at 63.64%	0.14	OK	Sec. G5
	1402	D9 at 63.64%	0.17	OK	Sec. G5
	1404	D2 at 50.00%	0.51	OK	Eq. H1.2-1
	1409	D2 at 76.56%	0.29	OK	Eq. H1.2-1
	1410	D2 at 76.56%	0.26	OK	Eq. H1.2-1
	1421	D2 at 55.09%	0.18	OK	Eq. H1.2-1
	1422	D2 at 50.00%	0.25	OK	Eq. H1.2-1
	1423	D9 at 77.78%	0.15	OK	Sec. G5
	1426	D2 at 70.00%	0.25	OK	Eq. H1.2-1
	1427	D2 at 50.00%	0.10	OK	Eq. H1.2-1
	1428	D2 at 67.67%	0.20	OK	Eq. H1.2-1
	1497	D2 at 63.64%	0.11	OK	Sec. G5
	1498	D2 at 63.64%	0.19	OK	Eq. H1.2-1
	1499	D2 at 63.64%	0.11	OK	Sec. G5
	1500	D2 at 63.64%	0.12	OK	Eq. H1.2-1
	1501	D2 at 63.64%	0.17	OK	Eq. H1.2-1
	1534	D2 at 63.64%	0.20	OK	Sec. G5
	1538	D2 at 63.64%	0.13	OK	Eq. H1.2-1
	1543	D8 at 70.00%	0.25	OK	Eq. H1.2-1

1544	D8 at 50.00%	0.11	OK	Eq. H1.2-1
1545	D2 at 70.00%	0.21	OK	Eq. H1.2-1
1562	D8 at 58.33%	0.22	OK	Eq. H1.2-1
1563	D8 at 37.50%	0.35	OK	Eq. H1.2-1
1564	D8 at 77.78%	0.16	OK	Eq. H1.2-1
1581	D8 at 31.82%	0.16	OK	Eq. H1.2-1
1582	D9 at 63.64%	0.17	OK	Sec. G5
1583	D8 at 33.33%	**0.68**	**OK**	Eq. H1.2-1
1612	D8 at 31.82%	0.16	OK	Eq. H1.2-1
1613	D8 at 63.64%	0.13	OK	Sec. G5
1614	D8 at 63.64%	0.13	OK	Sec. G5
1615	D8 at 63.64%	0.14	OK	Sec. G5
1616	D2 at 60.61%	0.21	OK	Eq. H1.2-1
1617	D2 at 63.64%	0.22	OK	Sec. G5
1618	D2 at 63.64%	0.39	OK	Sec. G5
1711	D2 at 63.64%	0.11	OK	Sec. G5
1714	D2 at 63.64%	0.12	OK	Eq. H1.2-1
1716	D8 at 63.64%	0.13	OK	Sec. G5
1718	D8 at 63.64%	0.13	OK	Sec. G5

RAM Elements **Salas O'Brien**

Current Date: 9/15/2026 5:52 PM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\Pump House Roof.retx

Steel Code Check
Concise Report

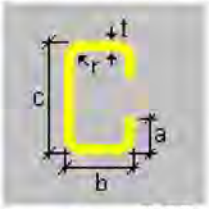
AISI 2016 LRFD (Cold-formed)

Member : 1398 (PEAK TRUSS) - OK

Section information

Section name: aisiSSMA_S 400S162-43 **(US)**

Dimensions



a	=	0.500	[in]	Lip
b	=	1.625	[in]	Flange width
c	=	4.000	[in]	Depth
r	=	0.071	[in]	Inside bend radius
t	=	0.045	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	0.357	
Moment of Inertia (local axes) (I)	[in4]	0.892	0.131
Moment of Inertia (principal axes) (I')	[in4]	0.892	0.131
Bending constant for moments (principal axis) (J')	[in]	0.000	2.250
Radius of gyration (local axes) (r)	[in]	1.581	0.606
Radius of gyration (principal axes) (r')	[in]	1.581	0.606
Saint-Venant torsion constant. (J)	[in4]	2.42E-04	
Section warping constant. (Cw)	[in6]	0.460	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-1.251	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	0.445	0.117
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	0.445	0.255
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	0.445	0.117
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	0.445	0.255
Plastic section modulus (local axis) (Z)	[in3]	0.517	0.174
Plastic section modulus (principal axis) (Z')	[in3]	0.517	0.174
Polar radius of gyration. (ro)	[in]	2.110	
Area for shear (Aw)	[in2]	0.143	0.221
Torsional constant. (C)	[in3]	0.005	

Material : A570 Gr50 cold form

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.30
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

Design Criteria

Description	Unit	Major axis	Minor axis
Additional hypotheses			
Full lateral restraints		No	
Continuous flexural torsional restraint		No	
Local axis design		No	
Region between inflection points adjacent to support		No	
Span type		Simple	
Fastened to support		Unfastened	
Local shear		No	
Braced for sidesway in major axis		No	
Braced for sidesway in minor axis		No	
Loading condition		EOF	
Flange support condition		Fastened	
Bearing length	[ft]	0.25	

Member lateral unbraced lengths

Length (Lb) [ft]		Restraint arrangement		Rotation restraint	
Top	Bottom	Top	Bottom	Top	Bottom
7.00	7.00	FF	FF	None	None

Compression member unbraced lengths

Length (L) [ft]			Effective length factor (ke)		
Major axis	Minor axis		Major axis	Minor axis	
7.00	7.00	7.00	0.00	0.00	1.0

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.07			
Capacity	:	16.07 [Kip]	Reference	:	Sec. D2
Demand	:	1.20 [Kip]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕT_n):	[Kip]	16.07	Sec. D2
Factored axial tensile fracture capacity(ϕT_n):	[Kip]	17.48	Sec. D3

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	3.58 [Kip]	Reference	:	Sec. G2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D14 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	3.58	Sec. G2

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	3.23 [Kip]	Reference	:	Sec. G2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	3.23	Sec. G2

Axial Compression Design ✓

Axial compression

Ratio	:	0.00			
Capacity	:	3.18 [Kip]	Reference	:	Sec. E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial compression capacity(ϕP_n):	[Kip]	3.42	Sec. E2
Factored axial compression distortional buckling capacity(ϕP_n):	[Kip]	10.23	Sec. E4
Factored local buckling strength(ϕP_n):	[Kip]	3.18	Sec. E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.00			
Capacity	:	0.91 [Kip*ft]	Reference	:	Sec. F2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D15 at 37.50%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	1.67	Sec. F2
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	0.91	Sec. F2
Factored moment distortional buckling capacity(ϕM_n):	[Kip*ft]	1.41	Sec. F4
Factored local buckling strength(ϕM_n):	[Kip*ft]	0.91	Sec. F3

Bending about the minor axis 22

Ratio	:	0.00			
Capacity	:	0.42 [Kip*ft]	Reference	:	Sec. F3
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D14 at 43.75%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	0.44	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	0.42	Sec. F3

Web Crippling Design ✔

Web crippling

Ratio	:	0.00			
Capacity	:	0.90 [Kip]	Reference	:	Sec. G5
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Factored web crippling capacity(ϕP_n):	[Kip]	0.90	Sec. G5

Combined Actions Design ✔

Combined bending and web crippling interaction

Ratio	:	0.00			
Ctrl Eq.	:	D15 at 0.00%	Reference	:	Sec. H3

Intermediate results	Unit	Value	Reference

Combined bending and shear interaction

Ratio	:	0.00			
Ctrl Eq.	:	D14 at 43.75%	Reference	:	Sec. H2

Intermediate results	Unit	Value	Reference
Combined bending and shear interaction (x-x):	--	0.00	
Combined bending and shear interaction(y-y):	--	0.00	Sec. H2

Combined bending and tension interaction

Ratio	:	0.07			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Eq. H1.1-1

Intermediate results	Unit	Value	Reference
Combined tensile axial and bending (flexure with tension yielding):	--	0.07	Eq. H1.1-1
Combined tensile axial and bending (flexure with buckling):	--	-0.07	

Combined bending and compression interaction

Ratio	:	0.00			
Ctrl Eq.	:	D14 at 43.75%	Reference	:	Eq. H1.2-1

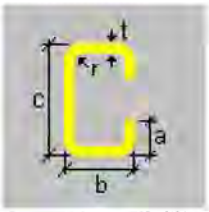
Intermediate results	Unit	Value	Reference
Combined bending and compression interaction:	—	0.00	Eq. H1.2-1

Member : 1406 (BOTTOM CHORD) - OK

Section information

Section name: aisiSSMA_S 1000S250-97 **(US)**

Dimensions



a	=	0.625	[in]	Lip
b	=	2.500	[in]	Flange width
c	=	10.000	[in]	Depth
r	=	0.153	[in]	Inside bend radius
t	=	0.102	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	1.580	
Moment of Inertia (local axes) (I)	[in4]	21.800	1.070
Moment of Inertia (principal axes) (I')	[in4]	21.800	1.070
Bending constant for moments (principal axis) (J')	[in]	0.000	6.030
Radius of gyration (local axes) (r)	[in]	3.714	0.823
Radius of gyration (principal axes) (r')	[in]	3.714	0.823
Saint-Venant torsion constant. (J)	[in4]	0.005	
Section warping constant. (Cw)	[in6]	21.600	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-1.454	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	4.348	0.553
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	4.348	1.866
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	4.348	0.553
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	4.348	1.866
Plastic section modulus (local axis) (Z)	[in3]	5.256	0.830
Plastic section modulus (principal axis) (Z')	[in3]	5.256	0.830
Polar radius of gyration. (ro)	[in]	4.080	
Area for shear (Aw)	[in2]	0.488	1.123
Torsional constant. (C)	[in3]	0.055	

Material : A570 Gr50 cold form

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.30
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

Design Criteria

Description	Unit	Major axis	Minor axis
Additional hypotheses			
Full lateral restraints		No	
Continuous flexural torsional restraint		No	
Local axis design		No	
Region between inflection points adjacent to support		No	
Span type		Simple	
Fastened to support		Unfastened	
Local shear		No	
Braced for sidesway in major axis		No	
Braced for sidesway in minor axis		No	
Loading condition		EOF	
Flange support condition		Fastened	
Bearing length	[ft]	0.67	

Member lateral unbraced lengths

Length (Lb) [ft]		Restraint arrangement		Rotation restraint	
Top	Bottom	Top	Bottom	Top	Bottom
3.50	3.50	FF	FF	None	None

Compression member unbraced lengths

Length (L) [ft]			Effective length factor (ke)		
Major axis	Minor axis		Major axis	Minor axis	
5.00	3.50	3.50	0.00	0.00	1.0

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.01			
Capacity	:	71.10 [Kip]	Reference	:	Sec. D2
Demand	:	1.04 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕT_n):	[Kip]	71.10	Sec. D2
Factored axial tensile fracture capacity(ϕT_n):	[Kip]	77.38	Sec. D3

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	11.54 [Kip]	Reference	:	Sec. G2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	11.54	Sec. G2

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	14.74 [Kip]	Reference	:	Sec. G2
Demand	:	-0.07 [Kip]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	14.74	Sec. G2

Axial Compression Design ✓

Axial compression

Ratio	:	0.00			
Capacity	:	33.46 [Kip]	Reference	:	Sec. E4
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial compression capacity(ϕP_n):	[Kip]	55.51	Sec. E2
Factored axial compression distortional buckling capacity(ϕP_n):	[Kip]	33.46	Sec. E4
Factored local buckling strength(ϕP_n):	[Kip]	36.51	Sec. E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.01			
Capacity	:	14.67 [Kip*ft]	Reference	:	Sec. F4
Demand	:	-0.21 [Kip*ft]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	16.31	Sec. F2
Factored moment distortional buckling capacity(ϕM_n):	[Kip*ft]	14.67	Sec. F4
Factored local buckling strength(ϕM_n):	[Kip*ft]	15.80	Sec. F3

Bending about the minor axis 22

Ratio	:	0.00			
Capacity	:	2.07 [Kip*ft]	Reference	:	Sec. F2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D9 at 100.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	2.07	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	2.07	Sec. F3

Web Crippling Design ✔

Web crippling

Ratio	:	0.54			
Capacity	:	4.82 [Kip]	Reference	:	Sec. G5
Demand	:	2.59 [Kip]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored web crippling capacity(ϕP_n):	[Kip]	4.82	Sec. G5

Combined Actions Design ✔

Combined bending and web crippling interaction

Ratio	:	0.29			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Sec. H3

Intermediate results	Unit	Value	Reference

Combined bending and shear interaction

Ratio	:	0.01			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Sec. H2

Intermediate results	Unit	Value	Reference
Combined bending and shear interaction (x-x):	--	0.01	Sec. H2
Combined bending and shear interaction(y-y):	--	0.00	

Combined bending and tension interaction

Ratio	:	0.03			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Eq. H1.1-1

Intermediate results	Unit	Value	Reference
Combined tensile axial and bending (flexure with tension yielding):	--	0.03	Eq. H1.1-1
Combined tensile axial and bending (flexure with buckling):	--	0.00	

Combined bending and compression interaction

Ratio	:	0.02			
Ctrl Eq.	:	D2 at 100.00%	Reference	:	Eq. H1.2-1

Intermediate results	Unit	Value	Reference
Combined bending and compression interaction:	—	0.02	Eq. H1.2-1

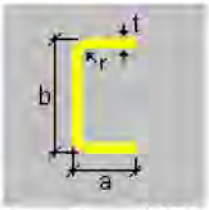
Member : 1412 (EAVE MEMBER) - With warnings

- Crippling design not applicable for built-up sections with unstiffened flanges fastened to support.
- Limit conditions to apply the reduction of bending strength of the member having one flange fastened are not complied.

Section information

Section name: aisiSSMA_T 1000T200-54 **(US)**

Dimensions



a	=	2.000	[in]	Flange width
b	=	10.198	[in]	Depth
r	=	0.085	[in]	Inside bend radius
t	=	0.057	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	0.792	
Moment of Inertia (local axes) (I)	[in4]	10.500	0.228
Moment of Inertia (principal axes) (I')	[in4]	10.500	0.228
Bending constant for moments (principal axis) (J')	[in]	0.000	7.200
Radius of gyration (local axes) (r)	[in]	3.641	0.537
Radius of gyration (principal axes) (r')	[in]	3.641	0.537
Saint-Venant torsion constant. (J)	[in4]	8.45E-04	
Section warping constant. (Cw)	[in6]	4.430	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-0.809	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	2.060	0.135
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	2.060	0.744
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	2.060	0.135
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	2.060	0.744
Plastic section modulus (local axis) (Z)	[in3]	2.556	0.224
Plastic section modulus (principal axis) (Z')	[in3]	2.556	0.224
Polar radius of gyration. (ro)	[in]	3.770	
Area for shear (Aw)	[in2]	0.223	0.574
Torsional constant. (C)	[in3]	0.015	

Material : A570 Gr50 cold form

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.30
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

Design Criteria

Description	Unit	Major axis	Minor axis
Additional hypotheses			
Full lateral restraints		No	
Continuous flexural torsional restraint		No	
Local axis design		No	
Region between inflection points adjacent to support		No	
Span type		Simple	
Fastened to support		Positive flange	
Local shear		No	
Braced for sidesway in major axis		No	
Braced for sidesway in minor axis		No	
Loading condition		EOF	
Flange support condition		Fastened	
Bearing length	[ft]	0.25	

Member lateral unbraced lengths

Length (Lb) [ft]		Restraint arrangement		Rotation restraint	
Top	Bottom	Top	Bottom	Top	Bottom
1.00	5.50	FF	FF	None	None

Compression member unbraced lengths

Length (L) [ft]			Effective length factor (ke)		
Major axis	Minor axis		Major axis	Minor axis	
5.50	5.50	5.50	0.00	0.00	1.0

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.01			
Capacity	:	35.64 [Kip]	Reference	:	Sec. D2
Demand	:	0.26 [Kip]	Ctrl Eq.	:	D8 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕT_n):	[Kip]	35.64	Sec. D2
Factored axial tensile fracture capacity(ϕT_n):	[Kip]	38.79	Sec. D3

Shear Design ✓

Shear in major axis 33

Ratio	:	0.01			
Capacity	:	6.00 [Kip]	Reference	:	Sec. G2
Demand	:	0.04 [Kip]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	6.00	Sec. G2

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	2.43 [Kip]	Reference	:	Sec. G2
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	2.43	Sec. G2

Axial Compression Design ✓

Axial compression

Ratio	:	0.00			
Capacity	:	5.41 [Kip]	Reference	:	Sec. E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial compression capacity(ϕP_n):	[Kip]	11.17	Sec. E2
Factored local buckling strength(ϕP_n):	[Kip]	5.41	Sec. E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.00			
Capacity	:	4.11 [Kip*ft]	Reference	:	Sec. F3
Demand	:	0.01 [Kip*ft]	Ctrl Eq.	:	D6 at 50.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	7.72	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	4.11	Sec. F3

Bending about the minor axis 22

Ratio	:	0.38			
Capacity	:	0.13 [Kip*ft]	Reference	:	Sec. F3
Demand	:	-0.05 [Kip*ft]	Ctrl Eq.	:	D7 at 50.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	0.51	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	0.13	Sec. F3

Web Crippling Design ✔

Web crippling

Ratio	:	0.01			
Capacity	:	0.90 [Kip]	Reference	:	Sec. G5
Demand	:	0.01 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Factored web crippling capacity(ϕP_n):	[Kip]	0.90	Sec. G5

Combined Actions Design ✔

Combined bending and web crippling interaction

Ratio	:	0.00			
Ctrl Eq.	:	D2 at 0.00%	Reference	:	Sec. H3

Intermediate results	Unit	Value	Reference

Combined bending and shear interaction

Ratio	:	0.38			
Ctrl Eq.	:	D7 at 50.00%	Reference	:	Sec. H2

Intermediate results	Unit	Value	Reference
Combined bending and shear interaction (x-x):	--	0.00	
Combined bending and shear interaction(y-y):	--	0.38	Sec. H2

Combined bending and tension interaction

Ratio	:	0.38			
Ctrl Eq.	:	D13 at 50.00%	Reference	:	Eq. H1.1-2

Intermediate results	Unit	Value	Reference
Combined tensile axial and bending (flexure with tension yielding):	--	0.02	
Combined tensile axial and bending (flexure with buckling):	--	0.38	Eq. H1.1-2

Combined bending and compression interaction

Ratio	:	0.38			
Ctrl Eq.	:	D7 at 50.00%	Reference	:	Eq. H1.2-1

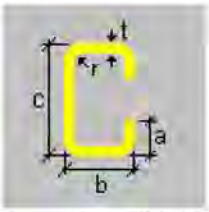
Intermediate results	Unit	Value	Reference
Combined bending and compression interaction:	—	0.38	Eq. H1.2-1

Member : 1484 (BRACE) - OK

Section information

Section name: aisiSSMA_S 400S162-43 **(US)**

Dimensions



a	=	0.500	[in]	Lip
b	=	1.625	[in]	Flange width
c	=	4.000	[in]	Depth
r	=	0.071	[in]	Inside bend radius
t	=	0.045	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	0.357	
Moment of Inertia (local axes) (I)	[in4]	0.892	0.131
Moment of Inertia (principal axes) (I')	[in4]	0.892	0.131
Bending constant for moments (principal axis) (J')	[in]	0.000	2.250
Radius of gyration (local axes) (r)	[in]	1.581	0.606
Radius of gyration (principal axes) (r')	[in]	1.581	0.606
Saint-Venant torsion constant. (J)	[in4]	2.42E-04	
Section warping constant. (Cw)	[in6]	0.460	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-1.251	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	0.445	0.117
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	0.445	0.255
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	0.445	0.117
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	0.445	0.255
Plastic section modulus (local axis) (Z)	[in3]	0.517	0.174
Plastic section modulus (principal axis) (Z')	[in3]	0.517	0.174
Polar radius of gyration. (ro)	[in]	2.110	
Area for shear (Aw)	[in2]	0.143	0.221
Torsional constant. (C)	[in3]	0.005	

Material : A570 Gr50 cold form

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.30
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

Design Criteria

Description	Unit	Major axis	Minor axis
Additional hypotheses			
Full lateral restraints		No	
Continuous flexural torsional restraint		No	
Local axis design		No	
Region between inflection points adjacent to support		No	
Span type		Simple	
Fastened to support		Unfastened	
Local shear		No	
Braced for sidesway in major axis		No	
Braced for sidesway in minor axis		No	
Loading condition		EOF	
Flange support condition		Fastened	
Bearing length	[ft]	0.25	

Member lateral unbraced lengths

Length (Lb) [ft]		Restraint arrangement		Rotation restraint	
Top	Bottom	Top	Bottom	Top	Bottom
6.31	6.31	FF	FF	None	None

Compression member unbraced lengths

Length (L) [ft]			Effective length factor (ke)		
Major axis	Minor axis		Major axis	Minor axis	
6.31	6.31	6.31	0.00	0.00	1.0

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.00			
Capacity	:	16.07 [Kip]	Reference	:	Sec. D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕT_n):	[Kip]	16.07	Sec. D2
Factored axial tensile fracture capacity(ϕT_n):	[Kip]	17.48	Sec. D3

Shear Design ✓

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	3.58 [Kip]	Reference	:	Sec. G2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D21 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	3.58	Sec. G2

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	3.23 [Kip]	Reference	:	Sec. G2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	3.23	Sec. G2

Axial Compression Design ✓

Axial compression

Ratio	:	0.18			
Capacity	:	3.72 [Kip]	Reference	:	Sec. E3
Demand	:	0.68 [Kip]	Ctrl Eq.	:	D9 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial compression capacity(ϕP_n):	[Kip]	4.11	Sec. E2
Factored axial compression distortional buckling capacity(ϕP_n):	[Kip]	10.23	Sec. E4
Factored local buckling strength(ϕP_n):	[Kip]	3.72	Sec. E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.00			
Capacity	:	1.08 [Kip*ft]	Reference	:	Sec. F2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D15 at 50.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	1.67	Sec. F2
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	1.08	Sec. F2
Factored moment distortional buckling capacity(ϕM_n):	[Kip*ft]	1.41	Sec. F4
Factored local buckling strength(ϕM_n):	[Kip*ft]	1.08	Sec. F3

Bending about the minor axis 22

Ratio	:	0.00			
Capacity	:	0.35 [Kip*ft]	Reference	:	Sec. F2
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D14 at 43.75%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	0.44	Sec. F2
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	0.35	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	0.35	Sec. F3

Web Crippling Design ✔

Web crippling

Ratio	:	0.00			
Capacity	:	0.90 [Kip]	Reference	:	Sec. G5
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Factored web crippling capacity(ϕP_n):	[Kip]	0.90	Sec. G5

Combined Actions Design ✔

Combined bending and web crippling interaction

Ratio	:	0.00			
Ctrl Eq.	:	D15 at 50.00%	Reference	:	Sec. H3

Intermediate results	Unit	Value	Reference

Combined bending and shear interaction

Ratio	:	0.00			
Ctrl Eq.	:	D15 at 50.00%	Reference	:	Sec. H2

Intermediate results	Unit	Value	Reference
Combined bending and shear interaction (x-x):	--	0.00	Sec. H2
Combined bending and shear interaction(y-y):	--	0.00	

Combined bending and tension interaction

Ratio	:	0.00			
Ctrl Eq.	:	D14 at 50.00%	Reference	:	Eq. H1.1-2

Intermediate results	Unit	Value	Reference
Combined tensile axial and bending (flexure with tension yielding):	--	0.00	Eq. H1.1-1
Combined tensile axial and bending (flexure with buckling):	--	0.00	Eq. H1.1-2

Combined bending and compression interaction

Ratio	:	0.18			
Ctrl Eq.	:	D9 at 43.75%	Reference	:	Eq. H1.2-1

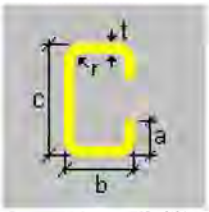
Intermediate results	Unit	Value	Reference
Combined bending and compression interaction:	—	0.18	Eq. H1.2-1

Member : 1583 (TOP CHORD) - OK

Section information

Section name: aisiSSMA_S 1000S250-97 **(US)**

Dimensions



a	=	0.625	[in]	Lip
b	=	2.500	[in]	Flange width
c	=	10.000	[in]	Depth
r	=	0.153	[in]	Inside bend radius
t	=	0.102	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	1.580	
Moment of Inertia (local axes) (I)	[in4]	21.800	1.070
Moment of Inertia (principal axes) (I')	[in4]	21.800	1.070
Bending constant for moments (principal axis) (J')	[in]	0.000	6.030
Radius of gyration (local axes) (r)	[in]	3.714	0.823
Radius of gyration (principal axes) (r')	[in]	3.714	0.823
Saint-Venant torsion constant. (J)	[in4]	0.005	
Section warping constant. (Cw)	[in6]	21.600	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-1.454	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	4.348	0.553
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	4.348	1.866
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	4.348	0.553
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	4.348	1.866
Plastic section modulus (local axis) (Z)	[in3]	5.256	0.830
Plastic section modulus (principal axis) (Z')	[in3]	5.256	0.830
Polar radius of gyration. (ro)	[in]	4.080	
Area for shear (Aw)	[in2]	0.488	1.123
Torsional constant. (C)	[in3]	0.055	

Material : A570 Gr50 cold form

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.30
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

Design Criteria

Description	Unit	Major axis	Minor axis
Additional hypotheses			
Full lateral restraints		No	
Continuous flexural torsional restraint		No	
Local axis design		No	
Region between inflection points adjacent to support		No	
Span type		Simple	
Fastened to support		Positive flange	
Local shear		No	
Braced for sidesway in major axis		Yes	
Braced for sidesway in minor axis		Yes	
Loading condition		EOF	
Flange support condition		Fastened	
Bearing length	[ft]	0.67	

Member lateral unbraced lengths

Length (Lb) [ft]		Restraint arrangement		Rotation restraint	
Top	Bottom	Top	Bottom	Top	Bottom
1.00	3.50	FF	FF	None	None

Compression member unbraced lengths

Length (L) [ft]			Effective length factor (ke)		
Major axis	Minor axis		Major axis	Minor axis	
3.50	3.50	21.00	3.50	0.00	1.0

Design Checks

Axial Tension Design ✔

Axial tension

Ratio : 0.00
Capacity : 71.10 [Kip]
Demand : 0.00 [Kip]
Reference : Sec. D2
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕT_n):	[Kip]	71.10	Sec. D2
Factored axial tensile fracture capacity(ϕT_n):	[Kip]	77.38	Sec. D3

Shear Design ✔

Shear in major axis 33

Ratio	:	0.01			
Capacity	:	11.54 [Kip]	Reference	:	Sec. G2
Demand	:	-0.08 [Kip]	Ctrl Eq.	:	D6 at 16.67%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	11.54	Sec. G2

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	19.18 [Kip]	Reference	:	Sec. G2
Demand	:	-0.14 [Kip]	Ctrl Eq.	:	D8 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	19.18	Sec. G2

Axial Compression Design ✓

Axial compression

Ratio	:	0.60			
Capacity	:	7.11 [Kip]	Reference	:	Sec. E3
Demand	:	4.27 [Kip]	Ctrl Eq.	:	D8 at 33.33%

Intermediate results	Unit	Value	Reference
Factored axial compression capacity(ϕP_n):	[Kip]	7.13	Sec. E2
Factored axial compression distortional buckling capacity(ϕP_n):	[Kip]	33.46	Sec. E4
Factored local buckling strength(ϕP_n):	[Kip]	7.11	Sec. E3

Flexural Design ✓

Bending about major axis, M33

Ratio	:	0.06			
Capacity	:	13.77 [Kip*ft]	Reference	:	Sec. F4
Demand	:	0.85 [Kip*ft]	Ctrl Eq.	:	D8 at 50.00%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	16.31	Sec. F2
Factored moment distortional buckling capacity(ϕM_n):	[Kip*ft]	13.77	Sec. F4
Factored local buckling strength(ϕM_n):	[Kip*ft]	15.80	Sec. F3

Bending about the minor axis 22

Ratio	:	0.08			
Capacity	:	0.99 [Kip*ft]	Reference	:	Sec. F2
Demand	:	-0.08 [Kip*ft]	Ctrl Eq.	:	D8 at 16.67%

Intermediate results	Unit	Value	Reference
Factored section moment capacity(ϕM_n):	[Kip*ft]	2.07	Sec. F2
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	0.99	Sec. F2
Factored local buckling strength(ϕM_n):	[Kip*ft]	0.99	Sec. F3

Web Crippling Design ✔

Web crippling

Ratio	:	0.03			
Capacity	:	4.82 [Kip]	Reference	:	Sec. G5
Demand	:	-0.14 [Kip]	Ctrl Eq.	:	D8 at 100.00%

Intermediate results	Unit	Value	Reference
Factored web crippling capacity(ϕP_n):	[Kip]	4.82	Sec. G5

Combined Actions Design ✔

Combined bending and web crippling interaction

Ratio	:	0.04			
Ctrl Eq.	:	D8 at 50.00%	Reference	:	Sec. H3

Intermediate results	Unit	Value	Reference

Combined bending and shear interaction

Ratio	:	0.05			
Ctrl Eq.	:	D8 at 50.00%	Reference	:	Sec. H2

Intermediate results	Unit	Value	Reference
Combined bending and shear interaction (x-x):	--	0.05	Sec. H2
Combined bending and shear interaction(y-y):	--	0.00	

Combined bending and tension interaction

Ratio	:	0.12			
Ctrl Eq.	:	D8 at 16.67%	Reference	:	Eq. H1.1-2

Intermediate results	Unit	Value	Reference
Combined tensile axial and bending (flexure with tension yielding):	--	0.04	Eq. H1.1-1
Combined tensile axial and bending (flexure with buckling):	--	0.12	Eq. H1.1-2

Combined bending and compression interaction

Ratio	:	0.68			
Ctrl Eq.	:	D8 at 33.33%	Reference	:	Eq. H1.2-1

Intermediate results	Unit	Value	Reference
Combined bending and compression interaction:	—	0.68	Eq. H1.2-1

ClarkDietrich Moment Clip

Moment connection for knee walls and shear walls to the structure foundation

ClarkDietrich moment clips are high-performance, cost effective solutions for knee wall-to-foundation connections and shear wall-to-foundation connections. These multi-application clips feature a 1/4" thick A36 steel stiffening plate that provides superior design values for maximum performance. The moment clips are designed to resist horizontal, torsional and vertical (uplift) loads. These clips are prepunched with a series of attachment holes for steel framing connections and a 1/2" anchor bolt hole for foundation connections.

PRODUCT DIMENSIONS

2" x 4" x 3-1/2"
2" x 4" x 5-1/2"
2" x 4" x 7-1/2"

1/4" A36 steel stiffening plate
1-3/4" x 3-3/8"
1-3/4" x 5-3/8"
1-3/4" x 7-3/8"

MATERIAL SPECIFICATIONS

Gauge: 14 gauge (68mils)
Design Thickness: 0.0713 inches

Gauge: 12 gauge (97mils)
Design Thickness: 0.1017 inches

Gauge: 10 gauge (118mils)
Design Thickness: 0.1242 inches

Coating: G90
Yield Strength: 50ksi
ASTM: A36, A653, A1003

INSTALLATION

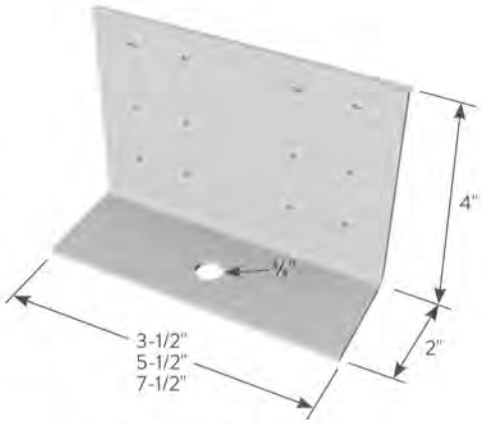
Secure the Moment Clip to the steel framing member by using #12 screws in the prepunched holes. Number of screws and screw pattern is based on load required to achieve listed capacities.

Place 1/4" steel stiffening plate on top of short leg of Moment Clip so anchor holes are aligned.

Secure 1/4" steel stiffening plate and Moment Clip to foundation using 1/2" x 2" long Hilti Kwik-Bolt 3 or alternate anchorage calculated to resist the tension load for your specific application.



1/4" Stiffening Plate



ClarkDietrich Moment Clip



ClarkDietrich Moment Clip w/ Stiffening Plate

ClarkDietrich Moment Clips (MC Series)

Clip Designation	Thickness		Size (in)	Packaging Pcs./Bucket
	mils (ga)	Design Thickness (in)		
MC683P	68 (14)	0.0713	2 x 4 x 3-1/2	25
MC685P	68 (14)	0.0713	2 x 4 x 5-1/2	25
MC687P	68 (14)	0.0713	2 x 4 x 7-1/2	25
MC973P	97 (12)	0.1017	2 x 4 x 3-1/2	25
MC975P	97 (12)	0.1017	2 x 4 x 5-1/2	25
MC977P	97 (12)	0.1017	2 x 4 x 7-1/2	25
MC103P	68 (14)	0.1242	2 x 4 x 3-1/2	25
MC105P	97 (12)	0.1242	2 x 4 x 5-1/2	25
MC107P	118 (10)	0.1242	2 x 4 x 7-1/2	25

Pub. No. CD-ClipExpress 01/26
The technical content of this literature is effective 01/08/26 and supersedes all previous information.

ClarkDietrich Moment Clips Allowable Loads

Clip Designation	Stud Member Thickness mils (ga)	Stud Fy (ksi)	F1 (Shear), (lbs) Number of #12 Screws to Stud			F2 (Tension), (lbs) Number of #12 Screws to Stud			M (Moment), (in-lbs) 1/2" dia. Anchor to Structure			Rotational Stiffness for Wind Deflection (in-lbs/radian)
			4	6	12	4	6	12	4	6	12	
MC683P 2" x 4" x 3-1/2" 68mils (14ga)	33 (20)	33	372	508	840	754	1131	2261[2]	1401	1784	2462[3]	133,000
	43 (18)	33	554	756	1250	1122	1683[2]	2736[2]	2084[3]	2581[3]	2581[3]	
	54 (16)	50	1126	1536	2086[1]	2278[2]	2736[2]	2736[2]	2743[3]	2743[3]	2743[3]	
	68 (14)	50	1535	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	2756[3]	2756[3]	2756[3]	
	97 (12)	50	1535	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	2756[3]	2756[3]	2756[3]	
MC973P 2" x 4" x 3-1/2" 97mils (12ga)	33 (20)	33	372	508	840	754	1131	2261[2]	1401	1784[3]	2560[3]	140,100
	43 (18)	33	554	756	1250	1122	1683[2]	3365[2]	2084[3]	2655	2862[3]	
	54 (16)	50	1126	1536	2538[1]	2278[2]	3417[2]	3832	2862[3]	2862[3]	2862[3]	
	68 (14)	50	1535	2094[1]	2975[1]	3107[2]	3832	3832	2862[3]	2862[3]	2862[3]	
	97 (12)	50	1535	2094[1]	2975[1]	3107[2]	3832	3832	2862[3]	2862[3]	2862[3]	
MC103P 2" x 4" x 3-1/2" 118mils (10ga)	33 (20)	33	372	508	840	754	1131	2261[2]	1401	1784[3]	2609[3]	143,900
	43 (18)	33	554	756	1250	1122	1683[2]	3365[2]	2084[3]	2655	2945[3]	
	54 (16)	50	1126	1536	2538[1]	2278[2]	3417[2]	3828[2]	2945[3]	2945[3]	2945[3]	
	68 (14)	50	1535	2094[1]	3462[1]	3107[2]	3828[2]	3828[2]	2945[3]	2945[3]	2945[3]	
	97 (12)	50	1535	2094[1]	3462[1]	3107[2]	3828[2]	3828[2]	2945[3]	2945[3]	2945[3]	
MC685P 2" x 4" x 5-1/2" 68mils (14ga)	33 (20)	33	441	630	1029	754	1131	2261[2]	1778	2417	3396[3]	252,000
	43 (18)	33	656	937	1531	1122	1683[2]	2736[2]	2646	3597[3]	4993[3]	
	54 (16)	50	1333	1903[1]	2086[1]	2278[2]	2736[2]	2736[2]	5307[3]	5307[3]	5307[3]	
	68 (14)	50	1818[1]	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	5363[3]	5363[3]	5363[3]	
	97 (12)	50	1818[1]	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	5363[3]	5363[3]	5363[3]	
MC975P 2" x 4" x 5-1/2" 97mils (12ga)	33 (20)	33	441	630	1029	754	1131	2261[2]	1778	2417	3396[3]	274,400
	43 (18)	33	656	937	1531	1122	1683[2]	3365[2]	2646	3597[3]	5025[3]	
	54 (16)	50	1333	1903[1]	2975[1]	2278[2]	3417[2]	4181[2]	5373[3]	5980[3]	5980[3]	
	68 (14)	50	1818[1]	2595[1]	2975[1]	3107[2]	4181[2]	4181[2]	5980[3]			
	97 (12)	50	1818[1]	2595[1]	2975[1]	3107[2]	4181[2]	4181[2]	5980[3]			
MC105P 2" x 4" x 5-1/2" 118mils (10ga)	33 (20)	33	441	630	1029	754	1131	2261[2]	1778			288,700
	43 (18)	33	656	937	1531	1122	1683[2]	3365[2]	2646			
	54 (16)	50	1333	1903[1]	3109[1]	2278[2]	3417[2]	4236[2]	5373[3]	6133[3]	6133[3]	
	68 (14)	50	1818[1]	2595[1]	3633[1]	3107[2]	4236[2]	4236[2]	6133[3]	6133[3]	6133[3]	
	97 (12)	50	1818[1]	2595[1]	3633[1]	3107[2]	4236[2]	4236[2]	6133[3]	6133[3]	6133[3]	
MC687P 2" x 4" x 7-1/2" 68mils (14ga)	43 (18)	33	791	1164	1928[1]	1122	1683[2]	2736[2]	3646	5199[3]	6903[3]	387,600
	54 (16)	50	1606	2086[1]	2086[1]	2278[2]	2736[2]	2736[2]	7403[3]	7656[3]	7656[3]	
	68 (14)	50	2086[1]	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	8186[3]	8186[3]	8186[3]	
	97 (12)	50	2086[1]	2086[1]	2086[1]	2736[2]	2736[2]	2736[2]	8186[3]	8186[3]	8186[3]	
MC977P 2" x 4" x 7-1/2" 97mils (12ga)	43 (18)	33	791	1164	1928[1]	1122	1683[2]	3365[2]	3646	5199[3]	6903[3]	449,000
	54 (16)	50	1606	2363[1]	2975[1]	2278[2]	3417[2]	3835[2]	7403[3]	8979[3]	8979[3]	
	68 (14)	50	2190[1]	2975[1]	2975[1]	3107[2]	3875[2]	3875[2]	8979[3]	8979[3]	8979[3]	
	97 (12)	50	2190[1]	2975[1]	2975[1]	3107[2]	3875[2]	3875[2]	8979[3]	8979[3]	8979[3]	
MC107P 2" x 4" x 7-1/2" 118mils (10ga)	43 (18)	33	791	1164	1928[1]	1122	1683[2]	3365[2]	3646	5199	6903[3]	451,000
	54 (16)	50	1606	2363[1]	3633[1]	2278[2]	3417[2]	3835[2]	7403[3]	9180[3]	9180[3]	
	68 (14)	50	2190[1]	3223[1]	3633[1]	3107[2]	4236[2]	4236[2]	9282[3]	9282[3]	9282[3]	
	97 (12)	50	2190[1]	3223[1]	3633[1]	3107[2]	4236[2]	4236[2]	9282[3]	9282[3]	9282[3]	

MAX ASD SHEAR = 2.88K
MAX ASD VERT = 2.6K DOWN 0.6K UPLIFT

SEE HILTI PROFIS OUTPUT. CLIP TO BE ATTACHED TO EMBED PLATE W/ CJP'D 1/2" DIA

Notes:

1 Cells marked with a " 1 " in shear column indicate that the shear capacity is limited to 1628 lbs when using 1/2" x 2" Hilti Kwik-Bolts into 3000psi concrete.

2 Cells marked with a " 2 " in tension column indicate that the tension capacity is limited to 1509 lbs when using 1/2" x 2" Hilti Kwik-Bolts into 3000psi concrete.

3 Cells marked with a " 3 " in the moment column indicate that the moment capacity is limited to 1761 in-lbs for 3-Series clips, 2767 in-lbs for 5-Series clips, and 3773 in-lbs for 7-Series clips when using 1/2" x 2" Hilti Kwik-Bolts into 3000psi concrete.

4 Capacities listed in the table/notes assume that no load reductions are required for spacing or edge distance of Kwik-Bolts.

5 Capacities listed in the table represent the capacity of the clip and the screws to the stud. Capacities listed in notes 1-3 are limits if the specified connector to the structure is used.

Other 1/2" dia. anchors may be used to achieve the full clip capacity but must be designed separately.

6 Moment capacities listed as Max. load/Factor of Safety. Loads must be limited by serviceability load taken as stiffness times the serviceability limit in radians.

7 Stiffness is the max. allowable clip moment divided by the clip rotation measured at the max. allowable clip moment.

8 No stud-to-track connection is required. Higher loads can be achieved when stud-to-track is connected.

9 Use a linear interaction equation for connections involving any combination of F1, F2, and M.

10 Allowable loads have not been increased for wind, seismic, or other factors.

11 Torsional effects are considered on screw group for F1 & F2 (Shear & Tension) allowable loads.

12 Hilti is a registered trademark of Hilti Aktiengeseilschaft Corporation.

13 It is the responsibility of the designer to properly detail connections on the contract drawings.





6 Screw Pattern



ClarkDietrich Moment Clip w/ 12 Screw Pattern & Anchor Bolt

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Pump House - Worst Case TOW Loading	Date:	9/15/2026
Fastening point:			

Specifier's comments:

1 Input data



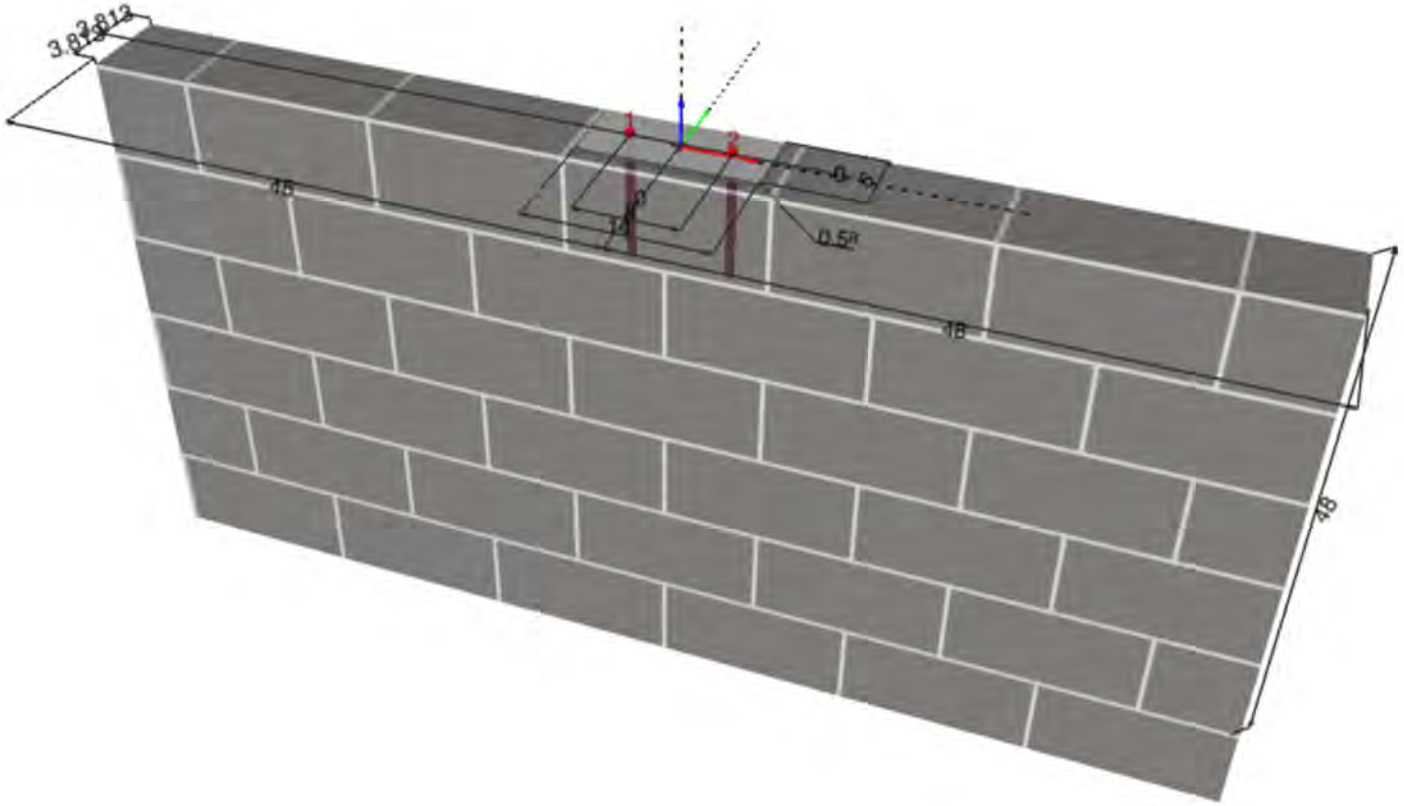
Anchor type and diameter:	**HY 270 + HAS-V-36 3/4**
Item number:	2198031 HAS-V-36 3/4"x12" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	HILTI ∅ 3/4 HY 270 + HAS-V-36 WITH 10 IN EFFECTIVE EMBEDMENT DEPTH PER ICC-ES ESR-4143 , HAMMER DRILLED INSTALLATION PER MPII
Effective embedment depth:	h_{ef} = 10.000 in.
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4143
Issued I Valid:	1/1/2026 | 1/1/2028
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-14
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R]:	l_x x l_y x t = 14.000 in. x 6.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	uncracked Grout-filled CMU, f=2000, f = 1,999 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Solid Head Joint: yes; open ended unit: yes
Temp. short/long:	32 °F / 32 °F
Installation:	Joints: vertical: 0.375 in.; horizontal: 0.375 in. Top installation, Drill hole: Hammer drilled, Installation condition: Dry

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
PROFIS Engineering (c) 2003-2026 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Pump House - Worst Case TOW Loading	Date:	9/15/2026
Fastening point:			

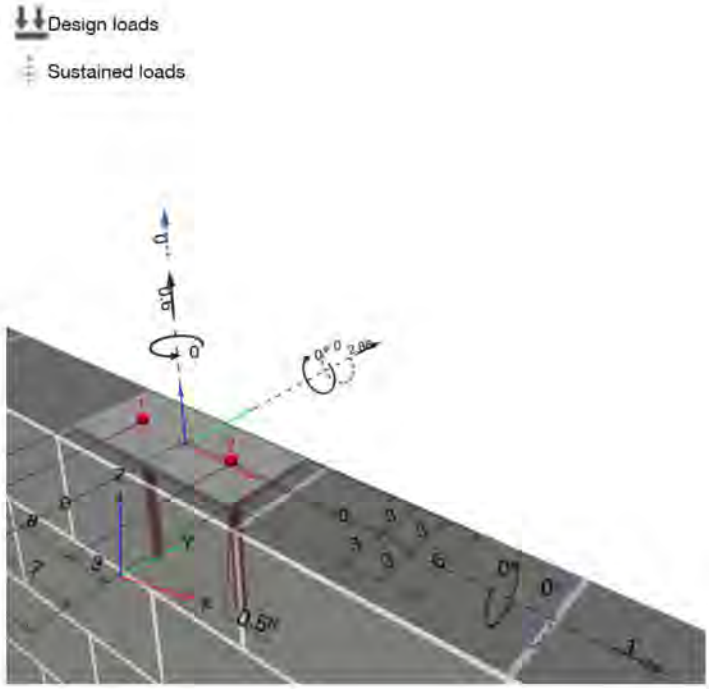
Geometry [in.]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Pump House - Worst Case TOW Loading	Date:	9/15/2026
Fastening point:			

Geometry [in.] & Loading [kip, in.lb]



1.1 Design results

Case	Description	Forces [kip] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0.600; V_x = 1.000; V_y = 2.880; M_x = 0; M_y = 0; M_z = 0; N_{sus} = 0.000; $M_{x,sus}$ = 0; $M_{y,sus}$ = 0;	no	75

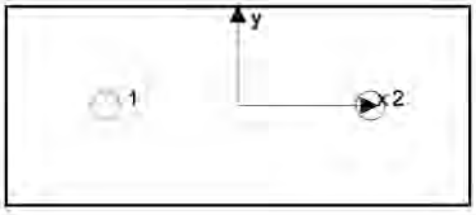
Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Pump House - Worst Case TOW Loading	Date:	9/15/2026
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2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.300	1.524	0.500	1.440
2	0.300	1.524	0.500	1.440



Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-/-): 0.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load [kip]	Capacity [kip]	Utilization β_N [%]	Status
Steel	0.300	14.549	3	OK
Bond	0.600	5.185	12	OK
Masonry breakout	0.600	3.904	16	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ESR-4143
$\phi\ N_{sa} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-14 Table 17.3.1.1

N_{sa} [kip]	ϕ	$\phi\ N_{sa}$ [kip]	N_{ua} [kip]
19.399	0.750	14.549	0.300

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3.2 Bond strength

$N_{mag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \cdot \psi_{ec1,Na} \cdot \psi_{ec2,Na} \cdot \psi_{ed,Na} \cdot N_{ba,m}$ — ACI 318-14 Eq. (17.4.5.1b)

$\phi\ N_{mag} \geq N_{ua}$ — AC58 Table 3.2 + ACI 318-14 Table 17.3.1.1

A_{Na} see ACI 318-14, Section 17.4.5.1, Fig. R 17.4.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ — ACI 318-14 Eq. (17.4.5.1c)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ — ACI 318-14 Eq. (17.4.5.1d)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N}{c_{Na}}}\right) \leq 1.00$ — ACI 318-14 Eq. (17.4.5.3)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.00$ — ACI 318-14 Eq. (17.4.5.4b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-14 Eq. (17.4.5.2)

Variables

d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	α_{sat}	α_{top}
0.750	10.000	3.812	200	1.000	1.000

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
3.184	81.11	40.55	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$N_{ba,m}$ [kip]
1.000	1.000	4.714

Results

N_{mag} [kip]	ϕ	$\phi\ N_{mag}$ [kip]	$\phi\ N_{ua}$ [kip]
9.428	0.550	5.185	0.600

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.3 Masonry breakout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec,N,m} \cdot \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ ACI 318-14 Eq. (17.4.2.1b)

$\phi \; N_{mbg} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-14 Table 17.3.1.1

A_{Nm} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N,m} = \left(\frac{1}{1 + \frac{2 \, e'_N}{3 \cdot h_{ef}}} \right) \leq 1.00$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f'_m} \cdot h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N,m}$ [in.]	$e_{c2,N,m}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
10.000	0.000	0.000	3.812	1.000	17

λ_a	f'_m [psi]
1.000	1,999

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N,m}$	$N_{b,m}$ [kip]
289.75	900.00	1.000	1.000	0.776	24.033

Results

N_{mbg} [kip]	ϕ	$\phi \; N_{mbg}$ [kip]	N_{ua} [kip]
6.006	0.650	3.904	0.600

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4 Shear load

	Load [kip]	Capacity [kip]	Utilization β_V [%]	Status
Steel	1.524	7.566	21	OK
Pryout cone	3.049	8.409	37	OK
Masonry crushing strength	1.524	4.449	35	OK
Masonry breakout in direction x+	3.049	4.104	75	OK

4.1 Steel strength

V_{sa} = ESR value — refer to ICC-ES ESR-4143

$\phi\ V_{sa} \geq V_{ua}$ — AC58 Table 3.2 + ACI 318-14 Table 17.3.1.1

V_{sa} [kip]	ϕ	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.639	0.650	7.566	1.524

4.2 Pryout strength (Masonry breakout strength controls)

$V_{mpg} = k_{cp} \cdot N_{mbg}$

$\phi\ V_{mpg} \geq V_{ua}$ — AC58 Table 3.2 + Section 8.7.2.2

A_{Nm} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.2a)

$$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c1,N}}{c_{Na}}}\right) \leq 1.0$$ — ACI 318-14 Eq. (17.4.2.4)

$$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c2,N}}{c_{Na}}}\right) \leq 1.0$$ — ACI 318-14 Eq. (17.4.2.4)

$$\psi_{ed,N,m} = 0.7 + 0.3\left(\frac{c}{c_{cr,N}}\right) \leq 1.0$$ — ACI 318-14 Eq. (17.4.2.5b)

$N_{b,m} = k_m\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	10.000	0.000	0.000	3.812	1.000

k_m	λ_a
17	1.000

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N}$	$N_{b,m}$ [kip]
289.75	900.00	1.000	1.000	0.776	24.033

Results

V_{mpg} [kip]	$\phi_{concrete}$	$\phi\ V_{mpg}$ [kip]	V_{ua} [kip]
12.012	0.700	8.409	3.049

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot (f'_m \cdot A_{se,V})^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
1,999	0.33

Results

V_{mc} [kip]	ϕ	$\phi\ V_{mc}$ [kip]	V_{ua} [kip]
8.899	0.500	4.449	1.524

4.4 Masonry breakout strength x+

$V_{mbg} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \Psi_{ec,V,m}\ \Psi_{ed,V,m}\ \Psi_{m,V}\ \Psi_{h,V,m}\ \Psi_{parallel,V}\ V_{b,m}$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{mbg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-14 Table 17.3.1.1

A_{Vm} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\Psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\Psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\Psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_{b,m} = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
6.000	0.750	32.000	3.812	366.00	4,608.00	1,999

Calculations

$\Psi_{ed,V,m}$	$\Psi_{parallel,V}$	$e_{c,V}$ [in.]	$\Psi_{ec,V,m}$	$\Psi_{m,V}$	$\Psi_{h,V,m}$
0.724	1.000	0.000	1.000	1.400	1.000

Results

$V_{b,m}$ [kip]	ϕ	$\phi\ V_{mbg}$ [kip]	V_{ua} [kip]
72.834	0.700	4.104	3.049

5 Required verifications under combined tension and shear forces

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.154	0.743	5/3	66	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor forces were determined under the assumption that the baseplate cannot deform under loading. The load distribution to the anchors is determined based on the assumption of linear elastic strains and strain compatibility between steel and concrete. No load re-distribution due to elastic deformations is considered. The proof that the rigid base plate assumption is valid is not carried out for the current design.
- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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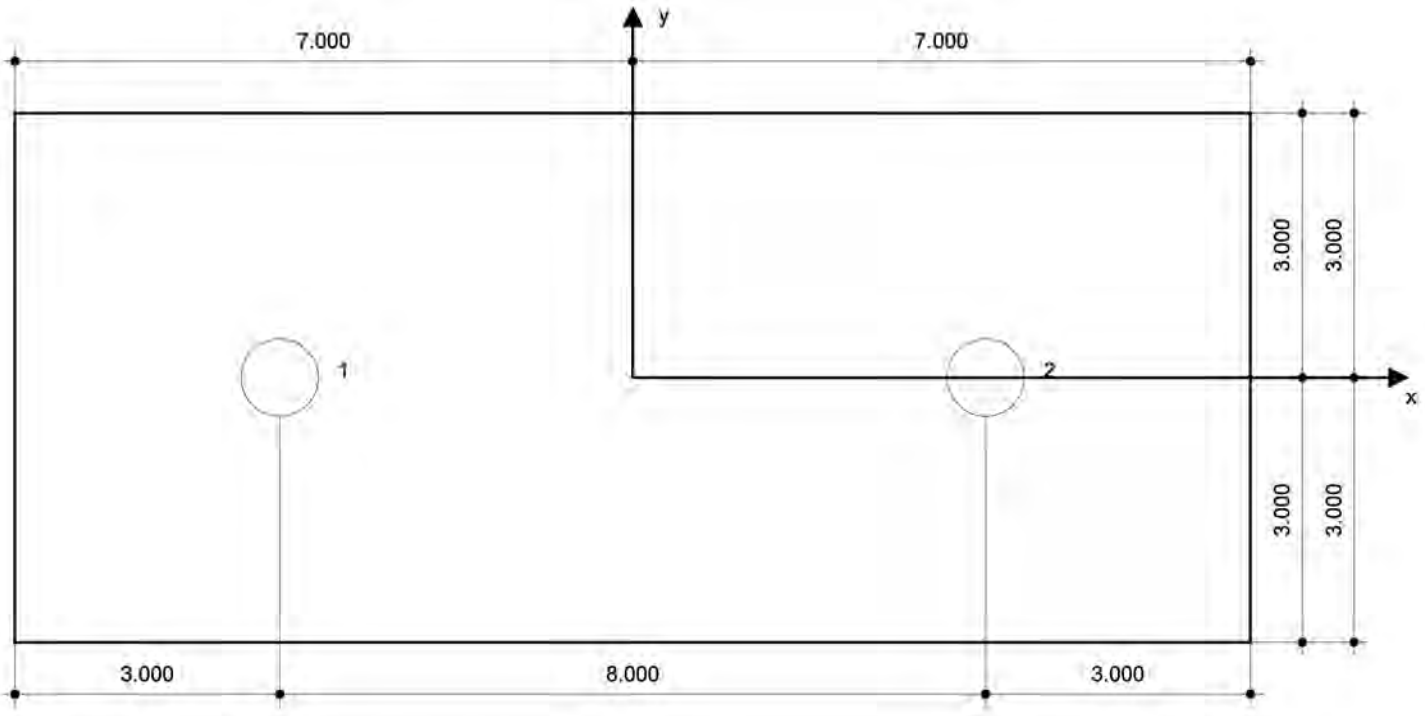
7 Installation data

Anchor plate, steel: ASTM A36; E = 29,000,001 psi; f_{yk} = 36,000 psi
Profile: no profile

Hole diameter in the fixture: d_f = 0.875 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: Drilled in hammer mode
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 270 + HAS-V-36 3/4
Item number: 2198031 HAS-V-36 3/4"x12" (element) / 2194247 HIT-HY 270 (adhesive)
Maximum installation torque: 120 in.lb
Hole diameter in the base material: 0.875 in.
Hole depth in the base material: 10.000 in.
Minimum thickness of the base material: 48.000 in.

Hilti ⌀ 3/4 HY 270 + HAS-V-36 with 10 in effective embedment depth per ICC-ES ESR-4143 , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.000	-0.000	44.000	52.000	3.812	3.812
2	4.000	-0.000	52.000	44.000	3.812	3.812

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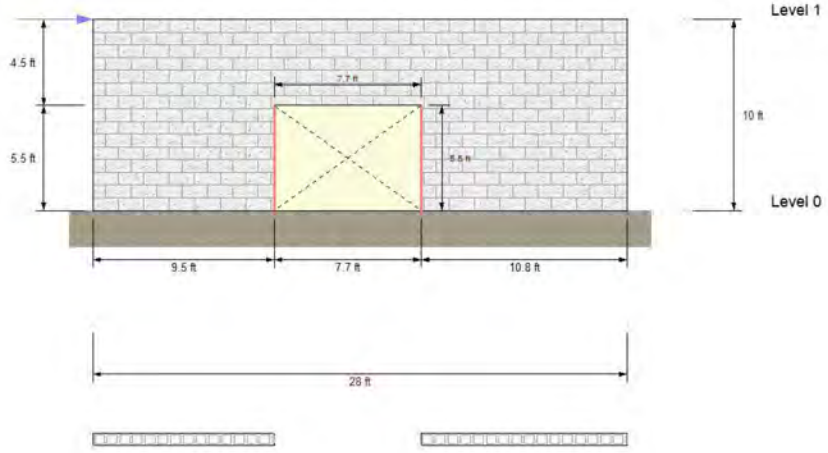
8 Remarks; Your Cooperation Duties

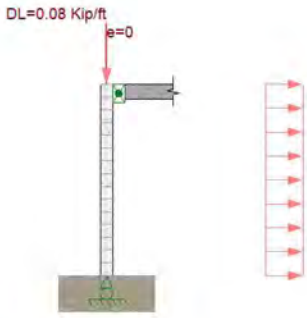
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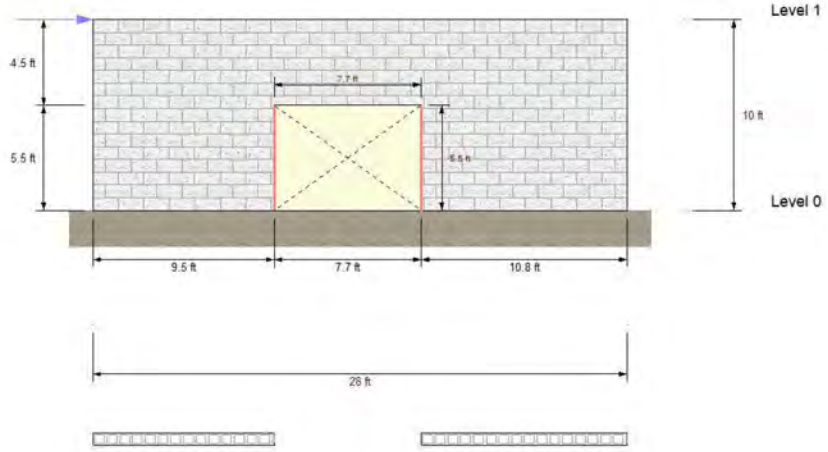
CMU WALL LOADING - DOOR OPENING

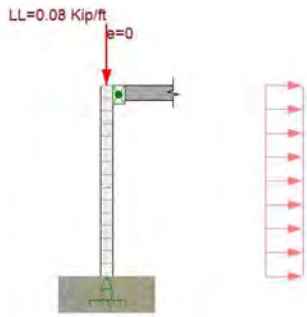
DEAD LOADS





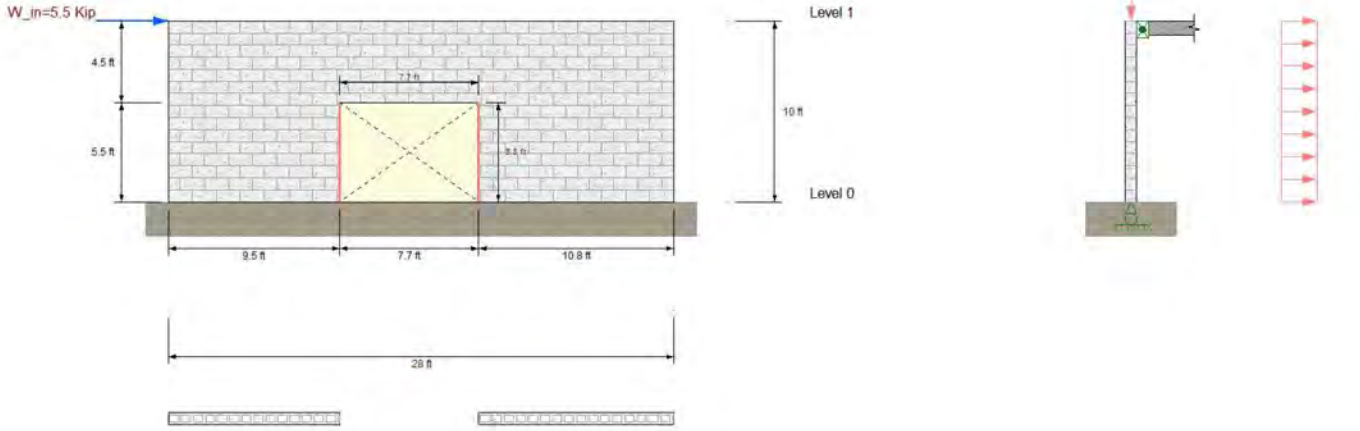
LIVE LOADS



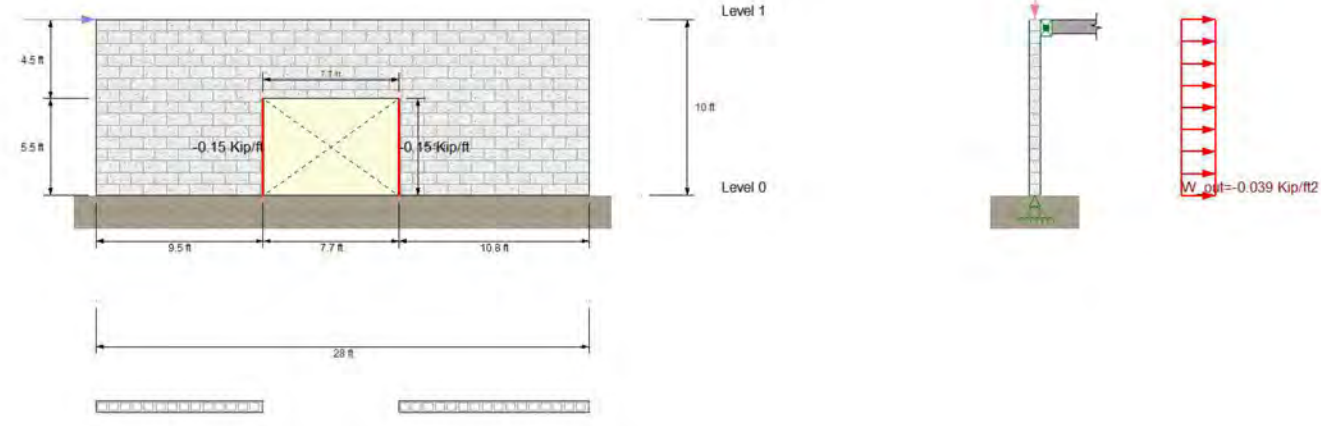


CMU WALL LOADING - DOOR OPENING

WIND - IN-PLANE SHEAR LOAD

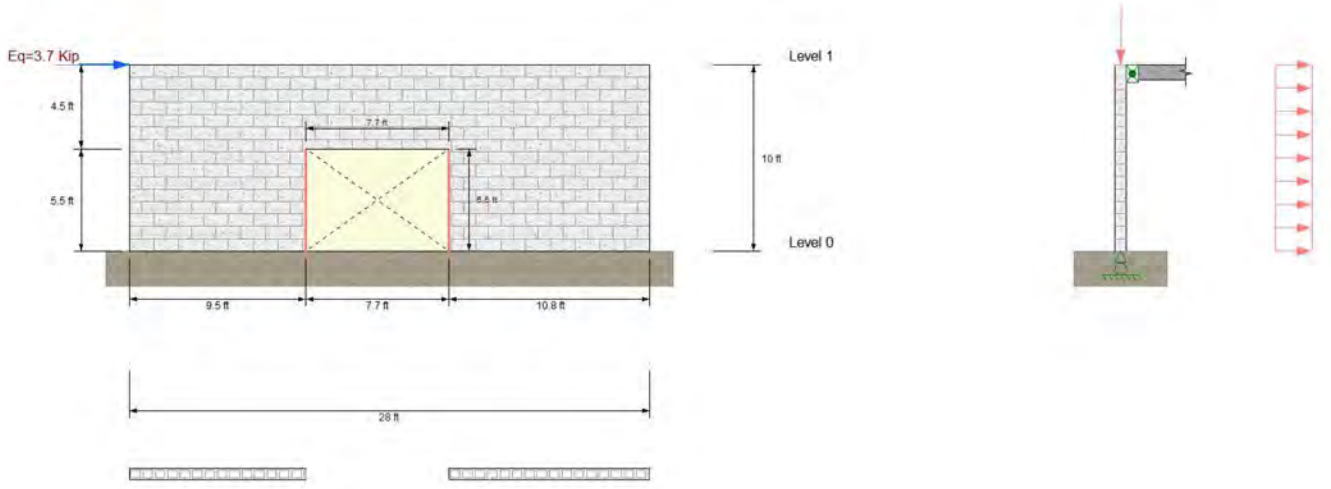


WIND - OUT-OF-PLANE SHEAR LOAD

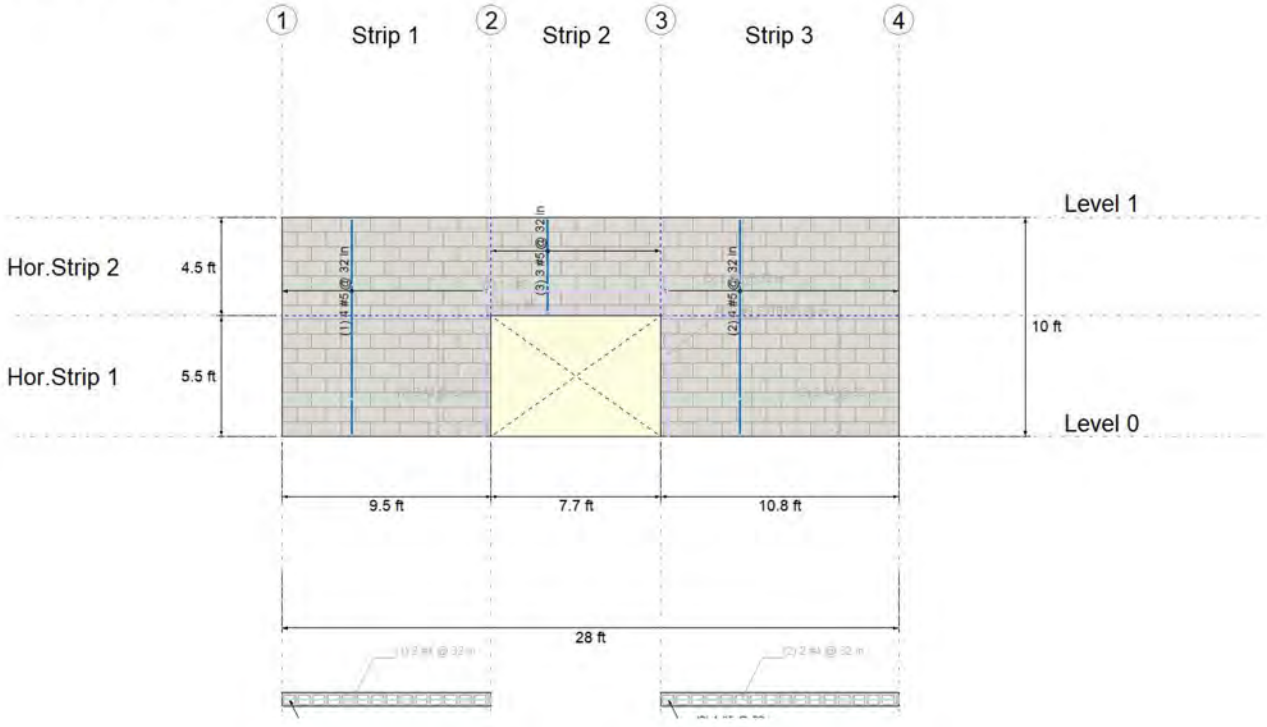


CMU WALL LOADING - DOOR OPENING

SEISMIC - EQ LOAD

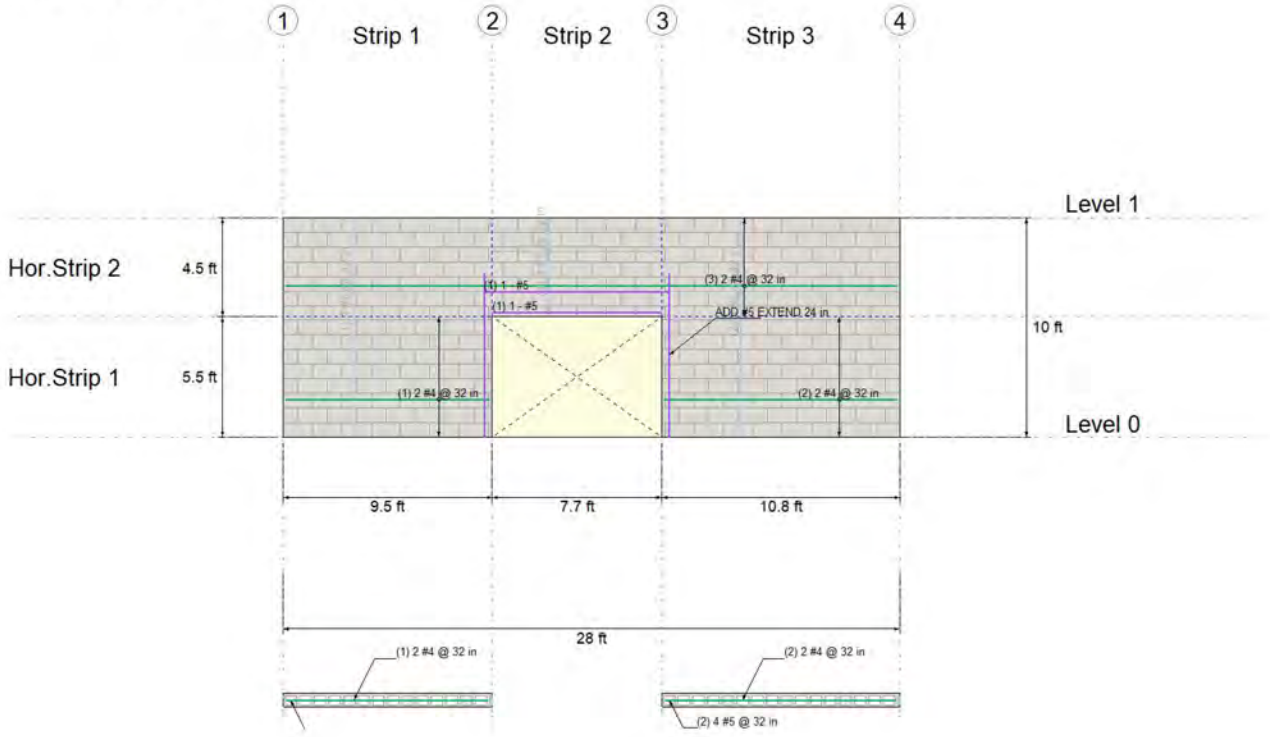


VERTICAL REINFORCEMENT



CMU WALL LOADING - DOOR OPENING

HORIZONTAL AND LINTEL REINFORCEMENT



RAM Elements **Salas O'Brien**

Current Date: 8/13/2026 10:50 AM
Units system: English
File name: G:\S99\103 AUBURNDALE\CALCS\00 MODELS\ELEMENTS\Pump House\CMU wall designs\8in CMU wall - Door Opening.msw

Design Results
Masonry wall

General Information

Global status : OK

Design code	:	TMS 402-16 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC A
Shear wall type	:	Ordinary

Geometry

Total height	:	10.00 [ft]
Total length	:	28.00 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	10.00	7.63	0.10

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	9.50	0.00	7.70	5.50

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
W_in	No	WIND	Wind In Plane
W_out	No	WIND	Wind Out of Plane
Eq	No	EQ	Seismic
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		DL
D2	Yes		DL+LL
D3	Yes		DL+0.75LL
D4	Yes		DL+0.6W_in
D5	Yes		DL+0.6W_out
D6	Yes		DL+0.75LL+0.45W_in
D7	Yes		DL+0.75LL+0.45W_out
D8	Yes		0.6DL+0.6W_in
D9	Yes		0.6DL+0.6W_out
D10	Yes		DL+0.7Eq
D11	Yes		DL+0.75LL+0.525Eq
D12	Yes		DL+0.525Eq
D13	Yes		0.6DL+0.7Eq
S1	Yes		DL
S2	Yes		DL+LL
S3	Yes		DL+0.75LL
S4	Yes		DL+0.6W_in
S5	Yes		DL+0.6W_out
S6	Yes		DL+0.7Eq
S7	Yes		DL+0.75LL+0.45W_in
S8	Yes		DL+0.75LL+0.45W_out
S9	Yes		0.6DL+0.6W_in
S10	Yes		0.6DL+0.6W_out
S11	Yes		DL+0.7Eq
S12	Yes		DL+0.75LL+0.525Eq
S13	Yes		DL+0.525Eq
S14	Yes		0.6DL+0.7Eq

Loads

Concentrated loads:

Story	Condition	Direction	Magnitude [Kip]	Eccentricity [in]	Distance [ft]
1	W_in	Horizontal	5.50	0.00	0.00
1	Eq	Horizontal	3.70	0.00	0.00

Distributed loads:

Consider self weight : DL

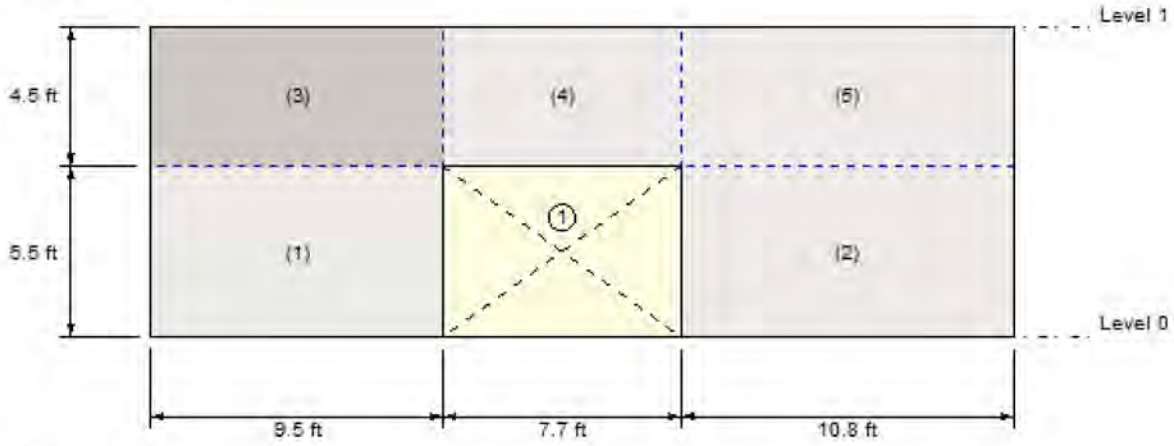
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	DL	Vertical	0.08	0.00
1	LL	Vertical	0.08	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	W_out	-0.04

Bearing Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	9.50	5.50
	2	17.20	0.00	10.80	5.50
	3	0.00	5.50	9.50	4.50
	4	9.50	5.50	7.70	4.50
	5	17.20	5.50	10.80	4.50

Vertical reinforcement

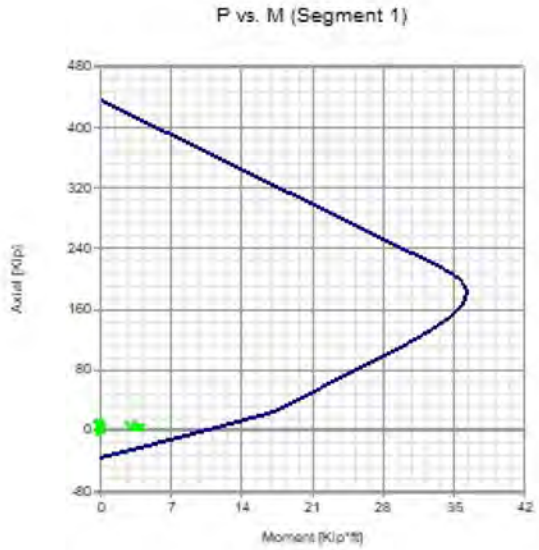
Reinforcement layers : 1

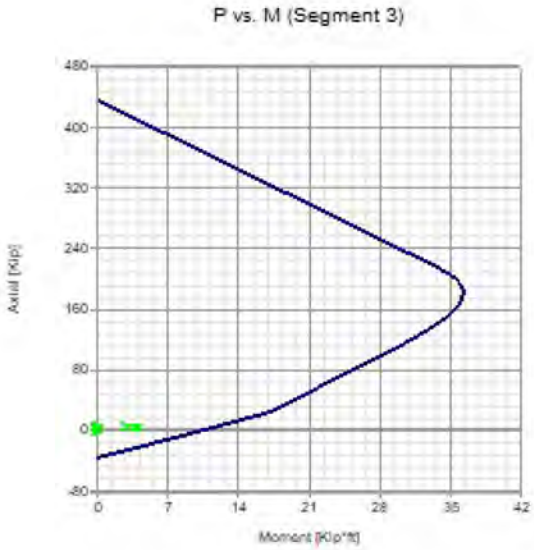
Segment	Bars	Spacing [in]	Ld [in]
1	4-#5	32.00	34.07
2	4-#5	32.00	34.07
3	4-#5	32.00	34.07
4	3-#5	32.00	34.07
5	4-#5	32.00	34.07

Combined axial flexure

Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D9(Top)	2.90	3.87	11.12	0.35
2	D9(Top)	3.19	4.25	12.61	0.34
3	D9(Max)	2.85	3.88	11.10	0.35
4	D9(Max)	0.85	1.13	8.59	0.13
5	D9(Max)	3.14	4.27	12.60	0.34

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D2(Bottom)	9.19	216.83	0.04
2	D6(Bottom)	10.29	246.51	0.04
3	D2(Bottom)	5.79	216.83	0.03
4	D7(Max)	1.84	175.75	0.01
5	D6(Bottom)	6.42	246.51	0.03

Axial tension

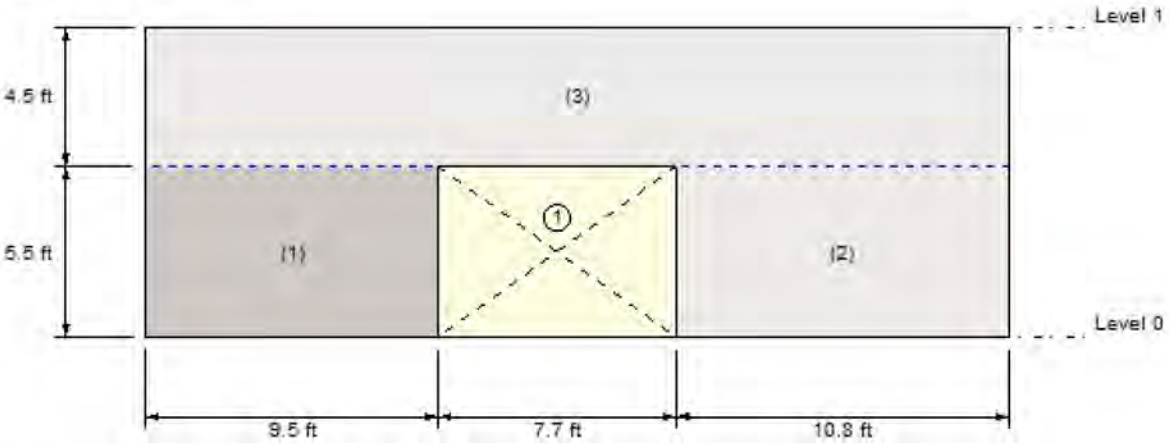
Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00
2	DM1(Top)	0.00	32.00	0.00
3	DM1(Top)	0.00	32.00	0.00
4	DM1(Top)	0.00	32.00	0.00
5	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D9(Max)	0.003	0.052	0.06
2	D9(Max)	0.003	0.052	0.06
3	D9(Top)	0.003	0.090	0.03
4	D9(Max)	0.002	0.051	0.03
5	D9(Top)	0.003	0.090	0.03

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	9.50	5.50
	2	17.20	0.00	10.80	5.50
	3	0.00	5.50	28.00	4.50

Reinforcement

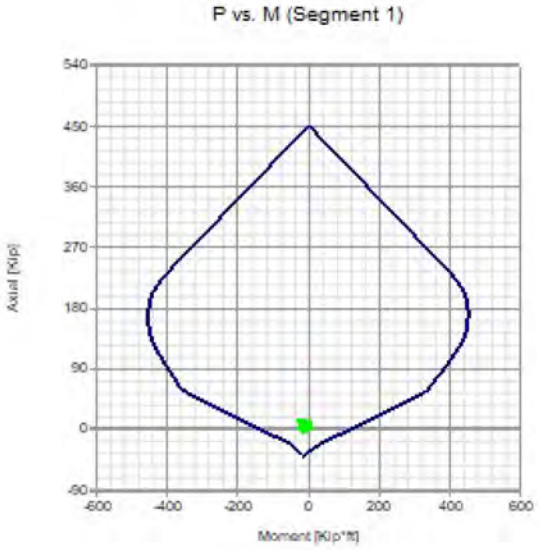
Reinforcement layers : 1

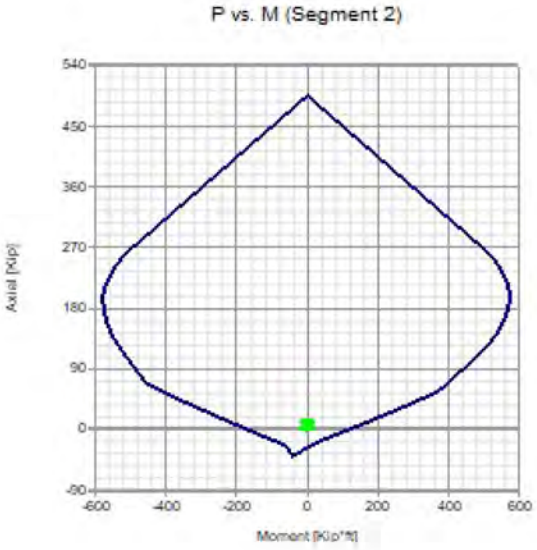
	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	4-#5	32.00	34.07	2-#4	32.00	21.80
2	4-#5	32.00	34.07	2-#4	32.00	21.80
3	4-#5	32.00	34.07	2-#4	32.00	21.80
	3-#5	32.00	34.07	2-#4	32.00	21.80
	4-#5	32.00	34.07	2-#4	32.00	21.80

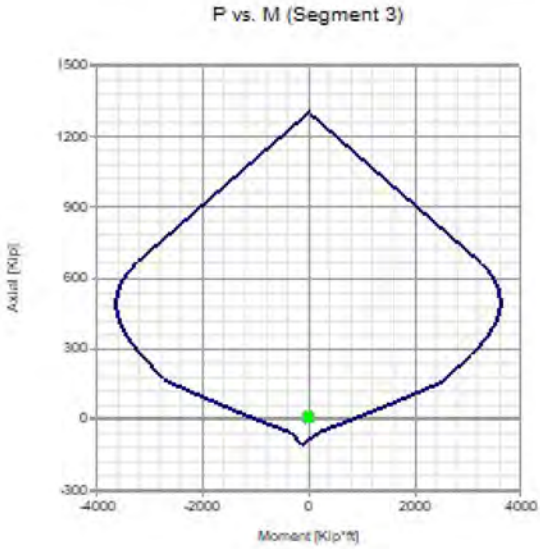
Combined axial flexure

Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D8(Bottom)	4.40	-18.04	154.77	0.12
2	D6(Top)	6.55	8.35	148.51	0.06
3	D8(Bottom)	6.09	-14.85	1053.52	0.01

Interaction diagrams, P vs. M







Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D2(Bottom)	9.19	223.74	0.04
2	D6(Bottom)	10.29	246.41	0.04
3	D2(Bottom)	12.40	646.40	0.02

Axial tension

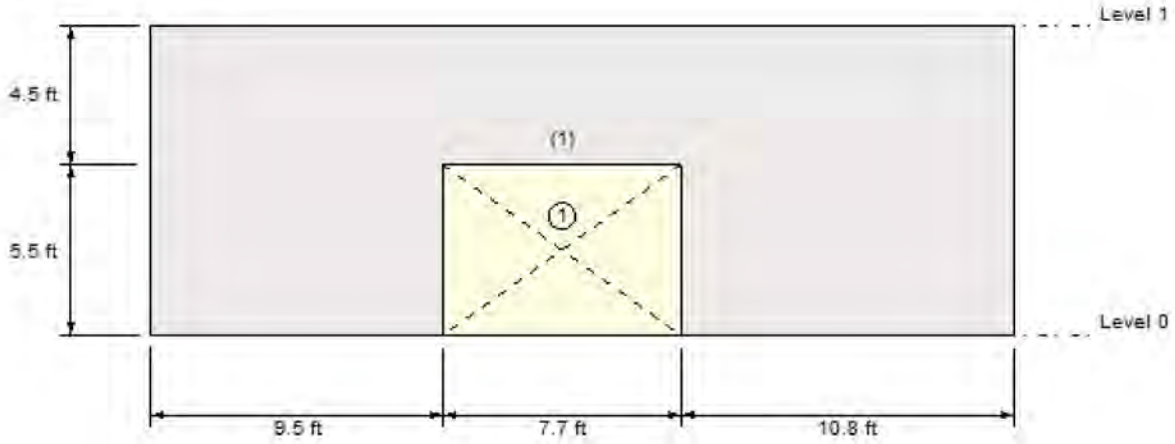
Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio	
1	DM1(Top)	0.00	32.00	0.00	
2	DM1(Top)	0.00	32.00	0.00	
3	DM1(Top)	0.00	32.00	0.00	

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio	
1	D8(Max)	0.003	0.056	0.05	
2	D4(Top)	0.004	0.077	0.05	
3	D8(Bottom)	0.002	0.081	0.03	

Lintel Design

Status : OK



Geometry

Lintel	X Coordinate [ft]	Y Coordinate [ft]	Length [ft]	Depth [in]
1	9.50	0.00	7.70	16.00

Reinforcement

Lintel	Top long. reinforcement Bars	Top long. reinforcement Extent [in]	Bottom long. reinforcement Bars	Bottom long. reinforcement Extent [in]	Transverse reinforcement Bars	Transverse reinforcement Spacing [in]	Ld [in]
1	1-#5	4.50	1-#5	0.00	--	0.00	0.00

Bending

Lintel	Condition	M [Kip*ft]	Ma [Kip*ft]	Ratio	
1	D4(Bottom)	3.42	10.31	0.33	

Shear

Lintel	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio	
1	D2(Top)	0.020	0.050	0.39	

Deflection

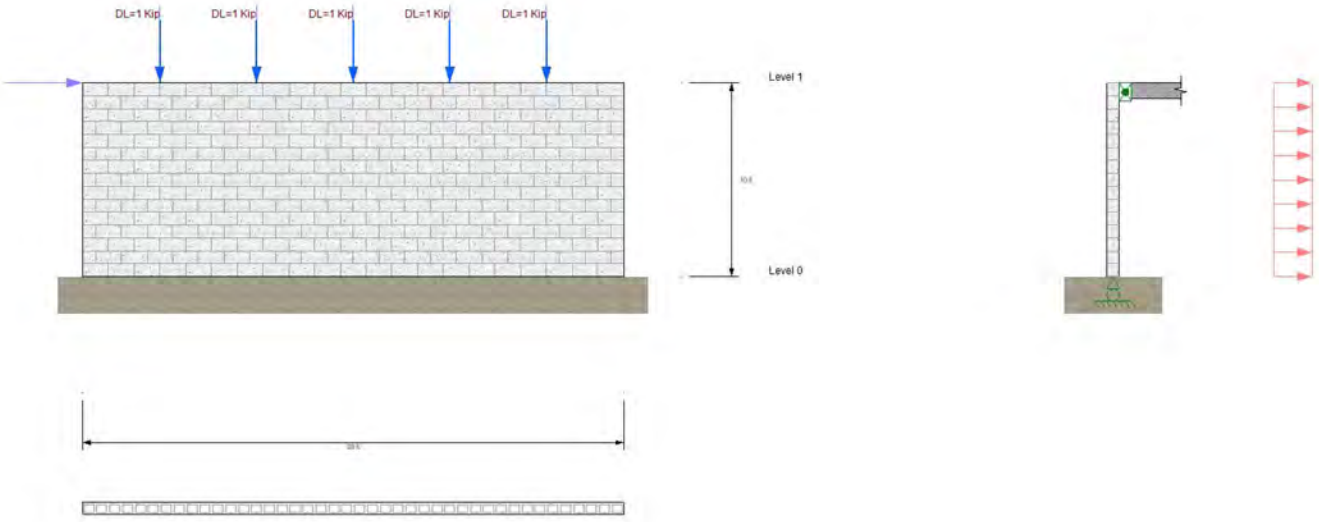
Lintel	Condition	δs [in]	δmax [in]	Ratio	
1		0.00	0.00	0.00	

Notes

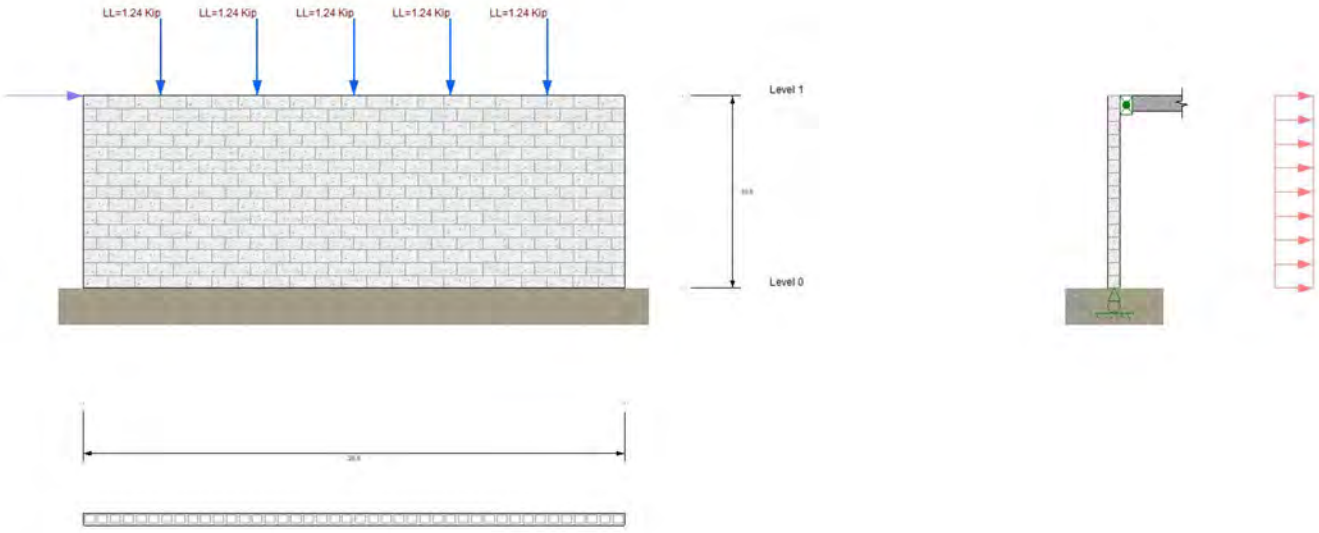
* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

CMU WALL LOADING - SOLID WALL W/ TRUSS LOADING

DEAD LOADS

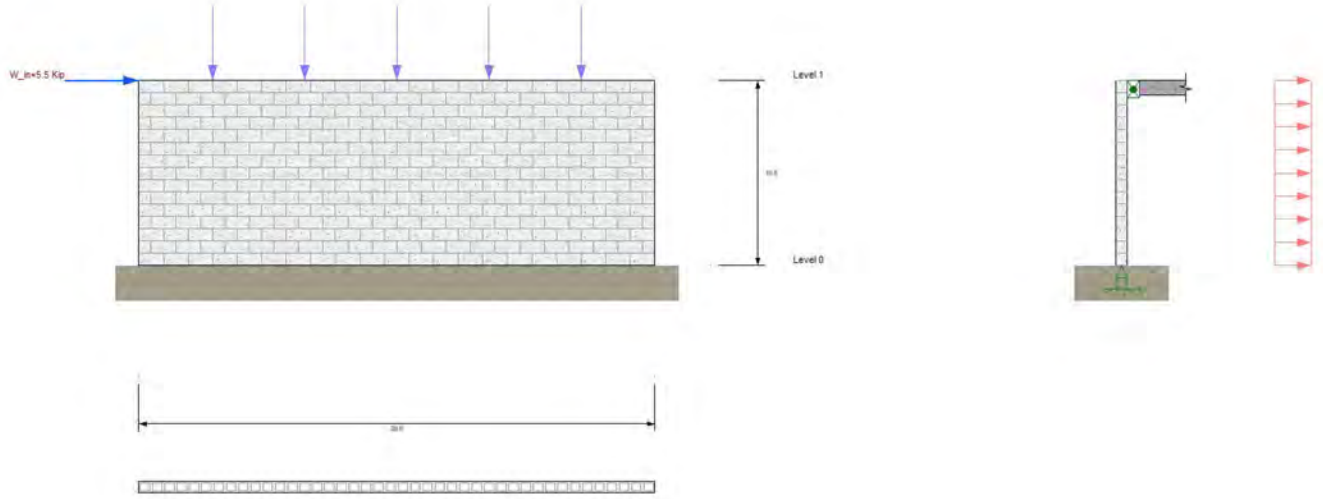


LIVE LOADS

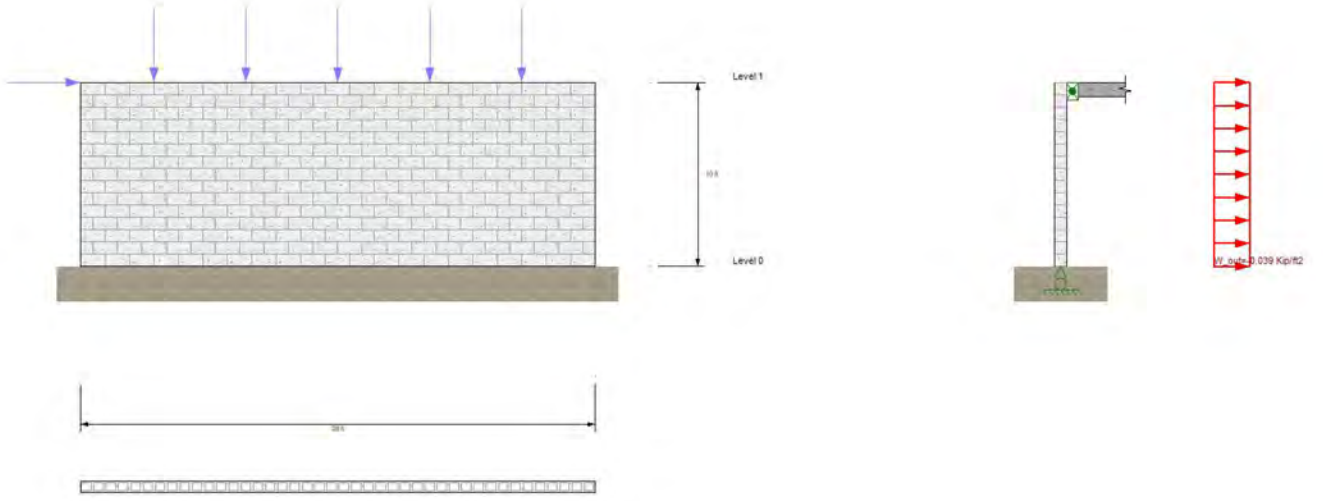


CMU WALL LOADING - SOLID WALL W/ TRUSS LOADING

WIND - IN-PLANE SHEAR LOAD

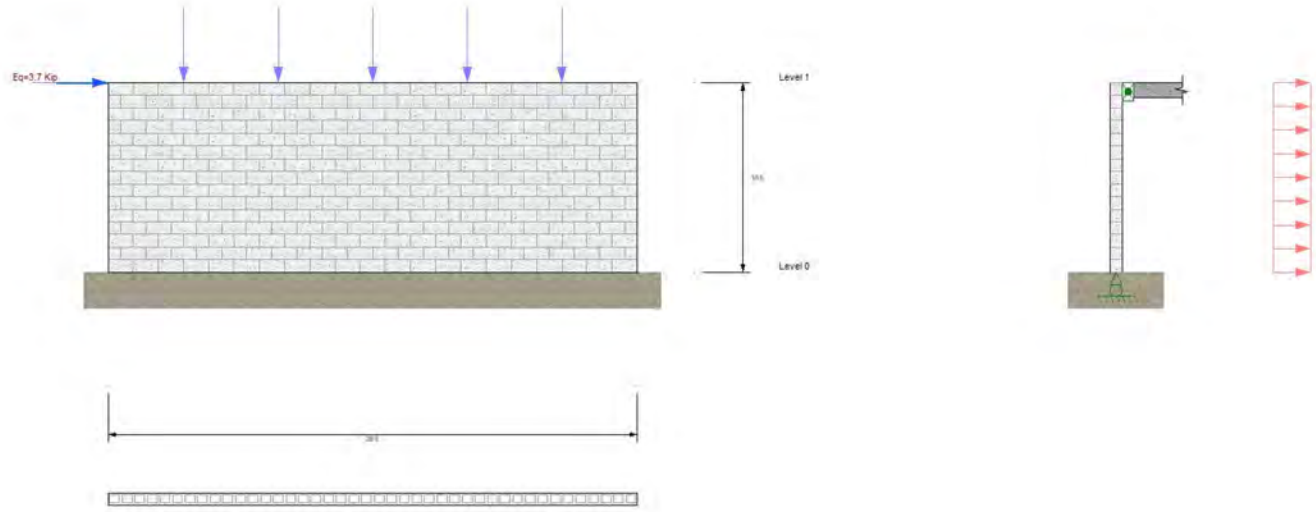


WIND - OUT-OF-PLANE SHEAR LOAD

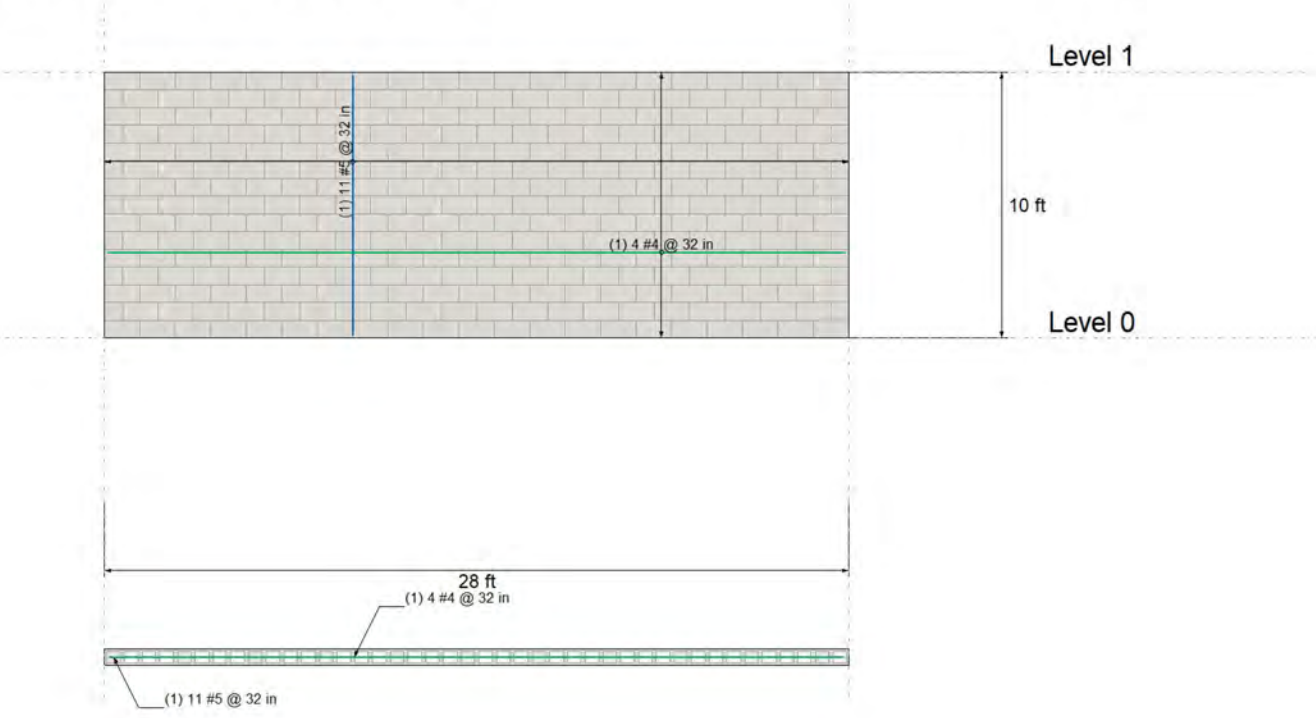


CMU WALL LOADING - SOLID WALL W/ TRUSS LOADING

SEISMIC - EQ LOAD



VERTICAL AND HORIZONTAL REINFORCEMENT



Current Date: 8/13/2026 10:48 AM
Units system: English
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Design Results
Masonry wall

General Information

Global status : OK

Design code	:	TMS 402-16 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC A
Shear wall type	:	Ordinary

Geometry

Total height	:	10.00 [ft]
Total length	:	28.00 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	10.00	7.63	0.10

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
W_in	No	WIND	Wind In Plane
W_out	No	WIND	Wind Out of Plane
Eq	No	EQ	Seismic
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		DL
D2	Yes		DL+LL
D3	Yes		DL+0.75LL
D4	Yes		DL+0.6W_in
D5	Yes		DL+0.6W_out
D6	Yes		DL+0.75LL+0.45W_in
D7	Yes		DL+0.75LL+0.45W_out
D8	Yes		0.6DL+0.6W_in
D9	Yes		0.6DL+0.6W_out
D10	Yes		DL+0.7Eq
D11	Yes		DL+0.75LL+0.525Eq
D12	Yes		DL+0.525Eq
D13	Yes		0.6DL+0.7Eq
S1	Yes		DL
S2	Yes		DL+LL
S3	Yes		DL+0.75LL
S4	Yes		DL+0.6W_in
S5	Yes		DL+0.6W_out
S6	Yes		DL+0.7Eq
S7	Yes		DL+0.75LL+0.45W_in
S8	Yes		DL+0.75LL+0.45W_out
S9	Yes		0.6DL+0.6W_in
S10	Yes		0.6DL+0.6W_out
S11	Yes		DL+0.7Eq
S12	Yes		DL+0.75LL+0.525Eq
S13	Yes		DL+0.525Eq
S14	Yes		0.6DL+0.7Eq

Loads

Concentrated loads:

Story	Condition	Direction	Magnitude [Kip]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	1.00	0.00	4.00
1	DL	Vertical	1.00	0.00	9.00
1	DL	Vertical	1.00	0.00	14.00
1	DL	Vertical	1.00	0.00	19.00
1	DL	Vertical	1.00	0.00	24.00
1	LL	Vertical	1.24	0.00	4.00
1	LL	Vertical	1.24	0.00	9.00
1	LL	Vertical	1.24	0.00	14.00
1	LL	Vertical	1.24	0.00	19.00
1	LL	Vertical	1.24	0.00	24.00
1	W_in	Horizontal	5.50	0.00	0.00
1	Eq	Horizontal	3.70	0.00	0.00

Distributed loads:

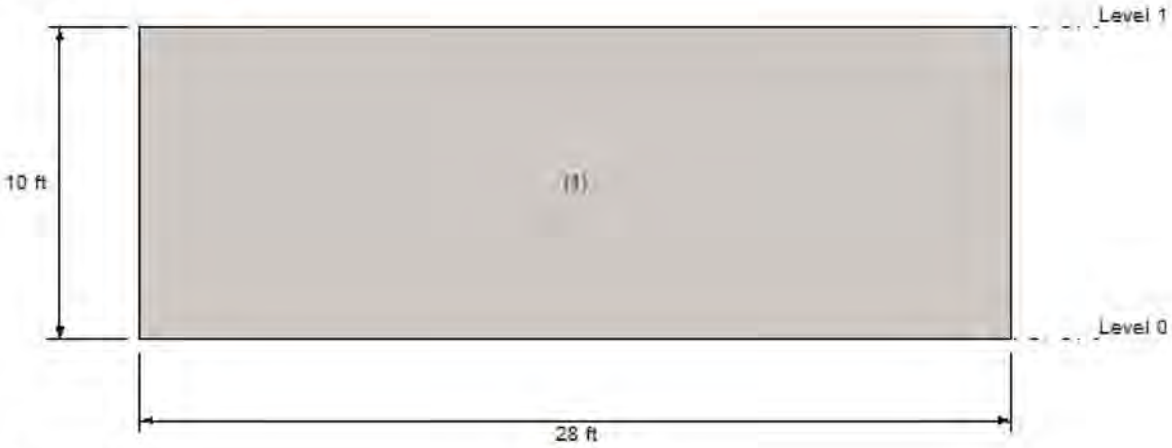
Consider self weight : DL

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	W_out	-0.04

Bearing Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	28.00	10.00

Vertical reinforcement

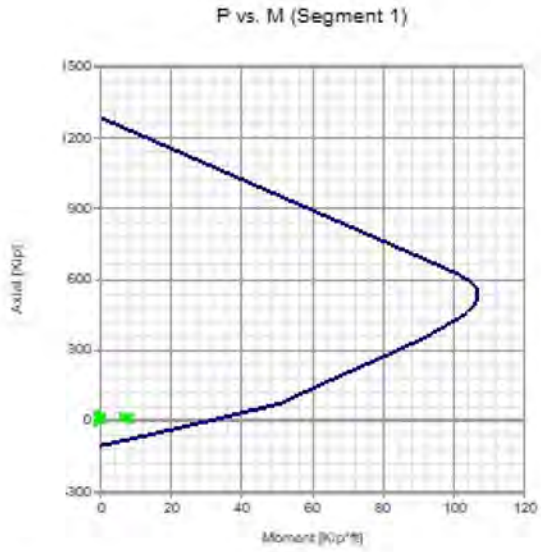
Reinforcement layers : 1

Segment	Bars	Spacing [in]	Ld [in]
1	11-#5	32.00	34.07

Combined axial flexure

Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D9(Max)	8.28	8.19	32.69	0.25

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio	
1	D2(Bottom)	28.79	639.09	0.05	

Axial tension

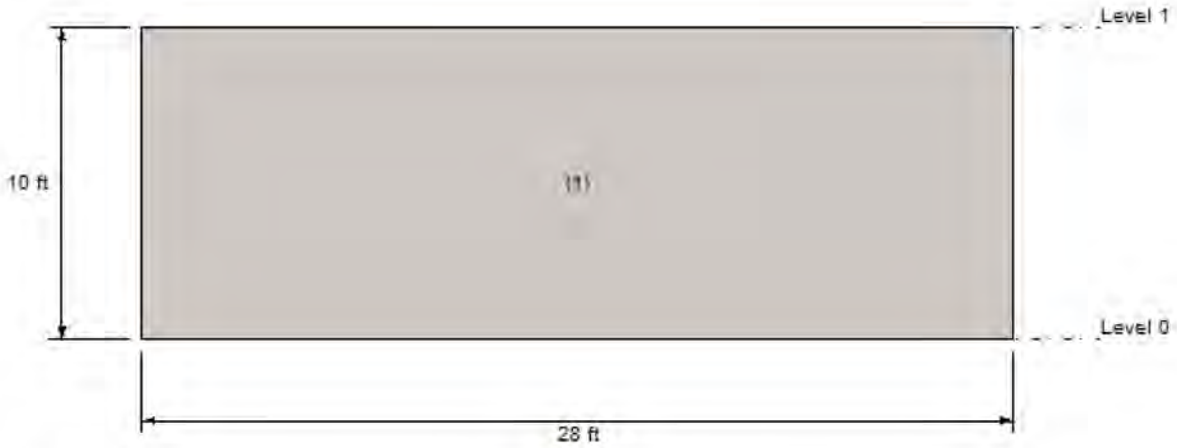
Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio	
1	DM1(Top)	0.00	32.00	0.00	

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio	
1	D9(Max)	0.002	0.052	0.03	

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	28.00	10.00

Reinforcement

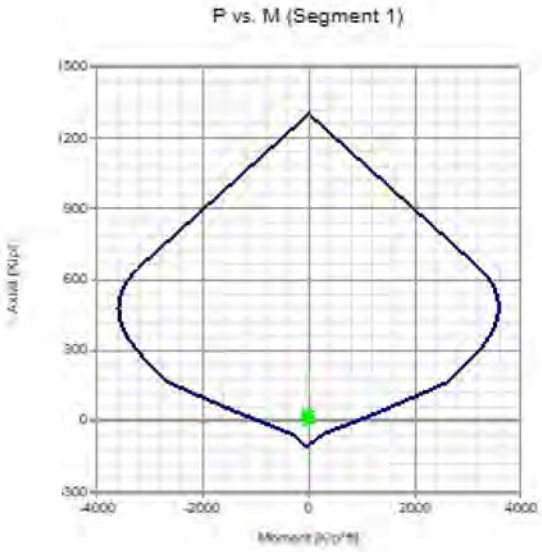
Reinforcement layers : 1

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	11-#5	32.00	34.07	4-#4	32.00	21.80

Combined axial flexure

Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D8(Bottom)	13.55	-33.00	1090.96	0.03

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D2(Bottom)	28.79	646.40	0.04

Axial tension

Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D8(Bottom)	0.002	0.076	0.03

Notes

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length

* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

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4700 LINCOLN ROAD NE SUITE 102 ALBUQUERQUE, NM 87109
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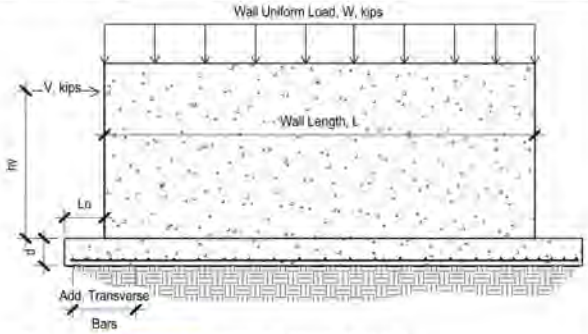
SHEET NO. 1 OF 3
PROJECT Project Sunshine
SUBJECT Pumphouse Shearwall Footing
CLIENT JOB NO. S99
CALCULATED BY LM DATE 8/13/2026
CHECKED BY DATE

Concrete Shearwall Footing Design

Input Criteria

Wall Criteria

Wall Length,L =	28.00	ft
Wall Thickness, h =	7.63	inches
Wall height to Shear Load, hv =	10.00	ft



Footing Criteria

Footing Ovehang Length, Lo =	0.00	ft
Footing Depth, d =	2.00	ft
Distance from top of grade to top of footing =	0.00	ft
Footing Width, W_f =	1.50	ft
Footing f'c =	3000	psi

Soil Criteria

Soil Coefficient of Friction, μ =	0.25		Reduction Factor:	1.00 (From Geotech Report)
Passive Pressure of Soil =	150.00	psf/ft	Reduction Factor:	0.50 (From Geotech Report)

Allowable Net Bearing Pressure =	3000	psf for Dead, Live Loads
Allowable Net Bearing Pressure =	3000	psf for Transient Loads (wind, Sesimic)

Soil weight = 115 lbs/ft^3, Overburden pressure = 0 psf

Load Criteria

Seismic Design Category:	A
Analysis Procedure:	Equivalent Lateral Force Procedure
California Building Code Applies?	No
Wind or Seismic Governs?	Wind
LLr or Snow Governs?	LLr
S_{DS} =	0.062
ρ =	1.0

LL Factor in LRFD Load Combos 3-5: 0.5

Input Loading

	Unfactored Loads						
	Self wt	DL	LL	LLr	S	W	Q_E
V, kips						5.50	3.70
W, kips/ft	1.24	0.32	0.00	0.32	0.00	0.00	

Sliding Resistance Criteria

Sliding Resistance avaialble from other sources: 0.00 kips
Source of Sliding resistance:

Footing Reinforcing:

Bottom Transverse Bars:	#5	bars @	8	in oc	s_{max} = 18 in OK
Top Transverse Bars:	#5	bars @	8	in oc	
Bottom Longitudinal Bars:	2	#5	continuous bars		
Top Longitudinal Bars:	2	#5	continuous bars		

Clear cover at bottom of footing =	3	in
Clear cover at side of footing =	3	in
Rebar Fy =	60	ksi

Summary of Results	
Bearing	Full Bearing
Overturning	0.07
Soil Press (Short-Term Loads)	0.45
footing bar development	Not Fully developed
1-way shear at boundaries	0.00
Flexural Design at boundaries	OK
Sliding	0.38
Static Load bearing Pressure	0.42
Static Load 1-way Shear	0.00
Static Load Moment	0.01
Minimum Flexural Reinforcing	OK
Flexural Reinf Max Spacing	OK
Temp & Shrinkage Reinf	OK

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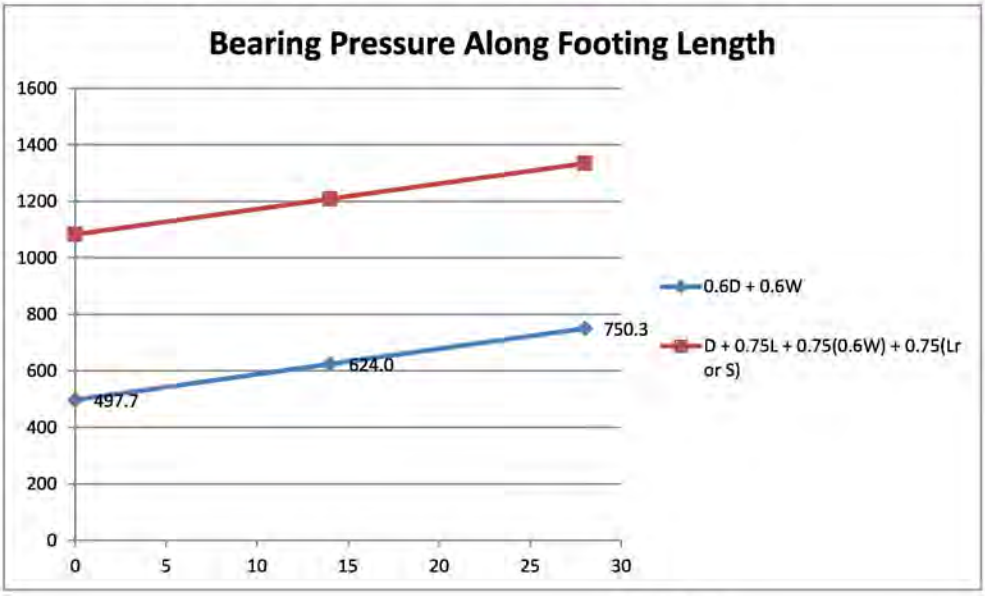
PROJECT Project Sunshine
SUBJECT Pumphouse Shearwall Footing
CLIENT 0 JOB NO. S99
CALCULATED BY LM DATE 8/13/2026
CHECKED BY 0 DATE 0

Concrete Shearwall Footing Design

	Load Factors					
LRFD Combos	DL	LL	LLr	S	W	Q_E
1.4D	1.4	0	0	0	0	0
1.2D + 1.6L + Lr	1.2	1.6	0.5	0	0	0
1.2D + 1.0W + 0.5L + 0.5Lr	1.2	0.5	0.5	0	1.0	0
1.2D + 1.0E + 0.5L + 0.2S	1.2124	0.5	0	0.2	0	1
0.9D + 1.0W	0.9	0	0	0	1.0	0
0.9D + 1.0E	0.8876	0	0	0	0	1
ASD Combos						
D + L	1.0	1.0	0	0	0	0
D + LLr	1.0	0	1.0	0.0	0	0
D + 0.75L + 0.75Lr	1.0	0.75	0.75	0	0	0
D + 0.6W	1.0	0	0	0	0.6	0
0.6D + 0.6W	0.6	0	0	0	0.6	0
D + 0.75L + 0.75(0.6W) + 0.75(Lr or S)	1.0087	0.75	0.75	0	0.45	0.525
0.6D + 0.7E	0.5913	0	0	0	0	0.7000

Calculate Overturning Effects

	Load Combo: 0.6D + 0.6W		Load Combo: D + 0.75L + 0.75(0.6W) + 0.75(Lr or S)	
Overall Footing Length, B =	28.00 ft			
Footing Area, A =	42.00 ft^2			
Footing Section Modulus =	196.00 ft^3			
Factored Overturning Moment, M_{u_OT} =	33.0 kip-ft		33.0	
Allowable Mu_OT Reduction per ASCE 7-10: 12.13.14	0.75			
Reduced Overturning Moment, M_{u_OT} =	24.8 kip-ft		24.8	
Factured Net Downward Load, P_{u_net} =	26.21 kip		50.8	
Total Overburden Pressure, P_{OB} =	0.00 kip		0.0	
Resisting Moment due to P_{OB} =	367 kip-ft		711 kip-ft	
Total Factored Downward Load, P_{u_gross} =	26.21 kip		50.78	
Resisting Moment due to P_{u_gross} =	0 kip-ft		0 kip-ft	
Factored Resisting Moment, M_R =	366.9 kip-ft		710.9	
Overturning Ratio M_R/M_{u_OT} =	0.07 OK		0.03 OK	
e = M/P =	0.94 ft		0.49 ft	
e_{crit} = L/6 =	4.67 ft	Full Bearing	4.67 ft	Full Bearing
Total Bearing Length, L_{brg} =	28.00 ft		28.00 ft	
Max Soil Bearing Pressure, q_{max} =	750 psf	Ratio: 0.25	1335 psf	Ratio: 0.45 OK
Minimum Soil Bearing Pressure, q_{min} =	498 psf		1083 psf	



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SHEET NO.	3	OF	3
PROJECT	Project Sunshine		
SUBJECT	Pumphouse Shearwall Footing		
CLIENT	0	JOB NO.	S99
CALCULATED BY	LM	DATE	8/13/2026
CHECKED BY	0	DATE	0

Concrete Shearwall Footing Design

Check Footing at Boundary Elements Load Combo: **1.2D + 1.0W + 0.5L + 0.5Lr** ϕ_v = 0.75, ϕ_m = 0.90

Factored Overturning Moment, M_{u_OT} =	**55.0** kip-ft		
Factured Net Downward Load, P_{u_net} =	**56.90** kip		
e = M/P =	**0.97** ft		
e_{crit} = L/6 =	**4.67** ft		
Total Bearing Length, L_{brg} =	**28.00** ft		
LRFD Factored Soil Bearing Pressure, Q_u =	**1635** psf		
d =	**20.69** in		
Dist to Moment Critical Section, L_{crit_M} =	**5.19** in		
Dist to Shear Critical Section, L_{crit_V} =	**0.00** in	**1-Way shear does not apply**	
M_u =	**1.83** kip-in per ft		
V_u =	**0.00** kips per ft		
available development length, l_{da} =	**9.81** in (at critical section for moment)		
required development length = l_d =	**41.08** in - flexural bars are	**Not Fully developed**	
Strength reduction factor for partially developed bars =	**0.24**		
1-way shear capacity, $\phi V_n = \phi 2(f'c)^{1/2} b_w d$ =	**20.40** kips/ft	Ratio: **0.00 OK**	
$K_u = M_u/b_w d^2$ =	**0.36**		
ρ_{req} =	**0.0000**		
ρ_{min} =	**0.0018**	Min Reinf: **OK**	[ACI 318: 10.5.4]
ρ_{req} =	**0.0018**		
ρ_{prov} =	**0.0019**	Ratio: **0.00 OK**	

Check Sliding Load Combo: **0.6D + 0.6W**

Total Factored Vertical Load =	**33.52** kips		
Total Factored Shear =	**3.30** kips		
Sliding Resistance due to Friction =	**8.38** kips		
Sliding Resistance due to Passive Pressure =	**0.23** kips		
Sliding Resistance from =	**0.00** kips		
Total Sliding Resistance =	**8.60** kips	Ratio: **0.38 OK**	

Check Bearing pressure for static loads

Load Combo	q, psf	ratio	OK/NG
D + L	**1040**	**0.35**	**OK**
D + LLr	**1253**	**0.42**	**OK**
D + 0.75L + 0.75Lr	**1200**	**0.40**	**OK**

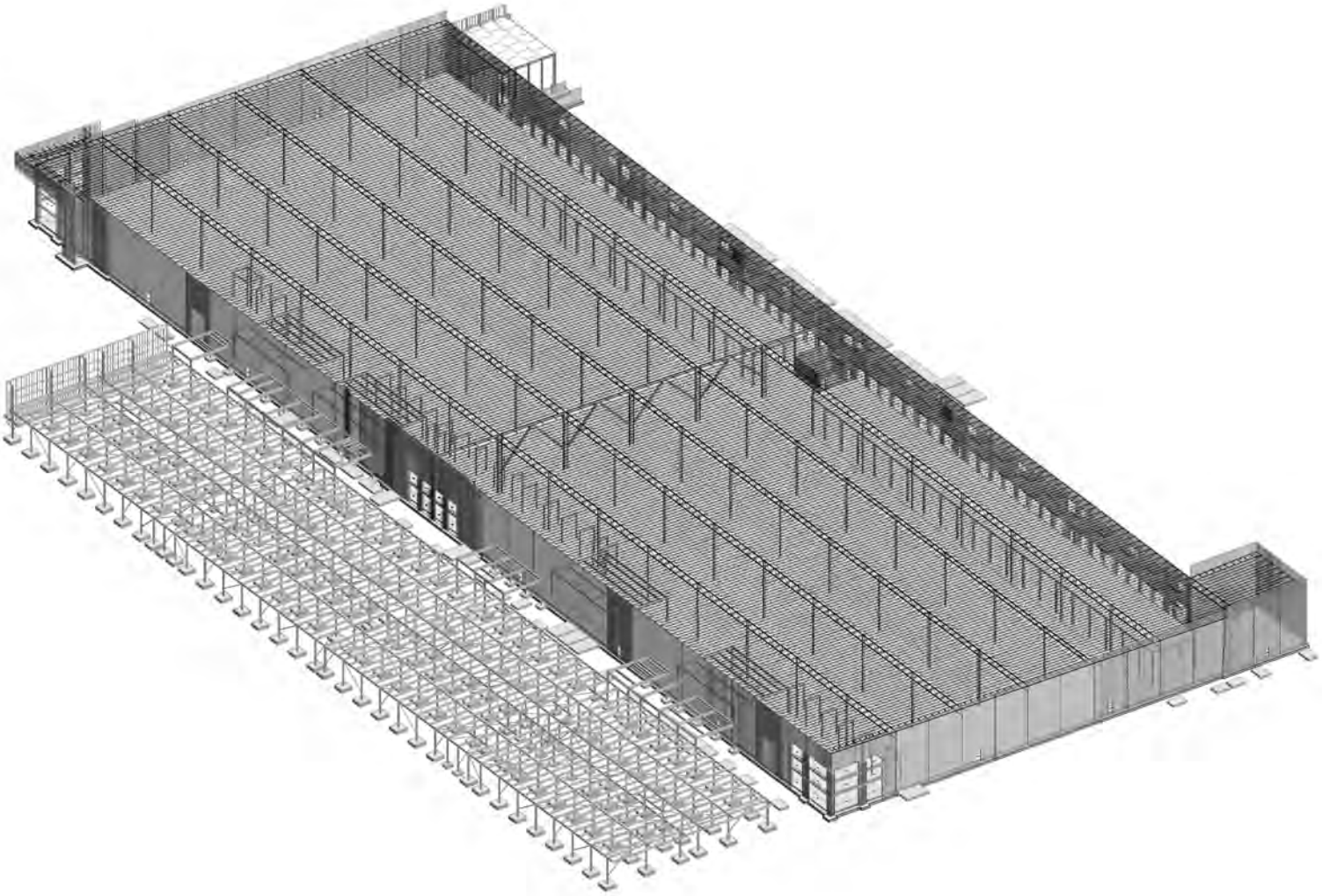
Check Transverse Reinforcing for static loads

Mu =	**1.41** kip-in per ft		
Vu =	**0.00** kips per ft		
$\phi V_n = \phi 2(f'c)^{1/2} b_w d$ =	**20.40** kips/ft	Ratio: **0.00 OK**	
$K_u = M_u/b_w d^2$ =	**0.27**		
ρ_{req} =	**0.0000**		
Controlling ρ_{req} =	**0.0018**		
ρ_{prov} =	**0.0019**	Ratio: **0.01 OK**	
		Min Reinf OK	

Check temperature Reinforcement

ρ_{min} =	**0.0018**		[ACI 318: 7.12.2]
ρ_{prov} =	**0.0029**	**OK**	

Chapter 9 - Existing Building Analysis with FM Wind Loads



AUG 14, 2026

STRUCTURAL CALCULATIONS

PROJECT SUNSHINE

Pace Rd adn Pearce Rd.
Auburndale, FL 33823

Axiom Job Number: A26-133

Prepared by:
Axiom PLLC
999 W Main St, Suite 200
Boise, ID 83702
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5.0 APPENDIX....103

DESIGN NARRATIVE

THE FOLLOWING CALCULATIONS ARE FOR CHECKING THE EXISTING BUILDING DESIGN AGAINST FM LOADING CRITERIA. THE BUILDING INQUESTION IS A 1 STORY WAREHOUSE FRAMED WITH OPEN WEB STEEL JOISTS, METAL DECK, AND CONCRETE TILT UP PANEL EXTERIOR WALLS.

1.0 DESIGN CRITERIA & LOADS

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SHEET NO.	1	OF	1
PROJECT	Auburndale		
SUBJECT	Wind Loads on Roof per FM Global		
CLIENT	Nuclei	JOB NO.	S99-103-26
CALCULATED BY	PD	DATE	5/1/2026
CHECKED BY		DATE	

Wind Loads on Roof per FM Global

Wind Pressure Based on FM Global:

			Reference
Velocity Pressure Coefficient, K_z =	1.04	Based on a mean roof height of 40'-10"	(FMDS0128, Table 3.2.2g)
Topographic Factor, K_{zt} =	1.0	Flat terrain	(FMDS0128, Section 3.2.1)
Directionality Factor, K_D =	0.85		(FMDS0128, Section 3.2.1)
Elevation Factor, K_E =	1.0	Elevation near sea level	(FMDS0128, Section 3.2.1)
Design Wind Speed, V =	100	mph	(FMDS0128, Appendix C Fig 3)
Importance Factor, I =	1.15		(FMDS0128, Section 3.2.1)

Wind Velocity Pressure, qh = $0.00256K_ZK_{Zt}K_DK_EV^2I$ = **26.02** psf

Wind Pressure for Specific Roof Zones

Building Mean Roof Height, h = 40.83 ft

	Zone 3	Zone 2	Zone 1	Zone 1'	
External Pressure Coefficient, GCp =	-3.2	-2.3	-1.7	-0.9	(FMDS0128, Table 3.2.2.2a)

Internal Pressure Coefficient, GCpi = 0.18 and -0.18 Enclosed building (FMDS0128, Section 3.2.1)

Design Wind Pressure, P = $[(GC_p) - (GC_{pi})]q_h$ = (FMDS0128, EQ 3.2.1b)

Safety Factor, SF = 2.0

Nominal Ultimate Design Wind Pressure, Pu = P(SF)

	+Gcpi	-Gcpi	+Gcpi	-Gcpi
	P, psf	-P, psf	+Pu, psf	-Pu, psf
Zone 3	-87.96	-78.60	**-175.93**	**-157.19**
Zone 2	-64.54	-55.17	**-129.08**	**-110.35**
Zone 1	-48.93	-39.56	**-97.85**	**-79.12**
Zone 1'	-28.11	-18.74	**-56.21**	**-37.48**

Zone 3 Length = 0.6h = 24.5 ft
Zone 3 Width = 0.2h = 8.17 ft
Zone 2 and 1 Width = 0.6h = 24.5 ft

3.2.2 Wind Zones of Buildings

This document provides calculation methods for wind loads on roofs and walls. It is extremely important to recognize that wind blowing over a roof exerts varying uplift forces on different areas of the roof and walls.

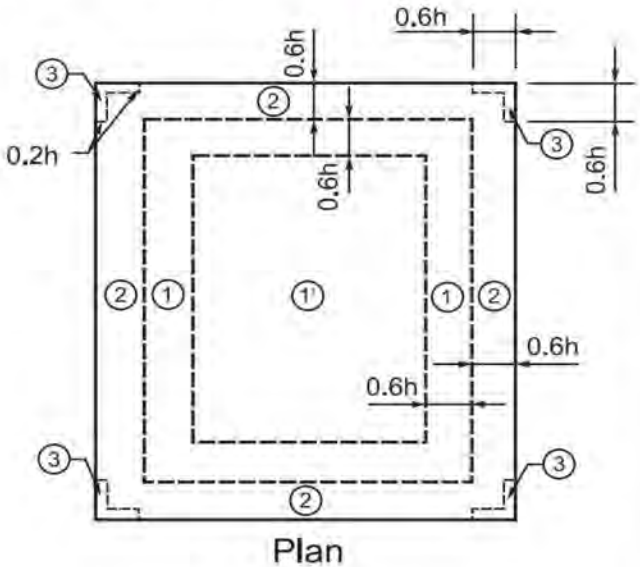
For low-slope roofs (≤7° and h ≤ 60 ft (18 m), or h < 90 ft (27 m) and h/w ≤1.0), the roof can be divided into four zones as a function of h, where h = the mean roof height. See Figure 3.2.2a and Table 3.2.2a for dimensions and coefficients.

Zone 3: The roof corners, with L-shaped dimension = 0.6h by 0.6h by 0.2h.

Zone 2: The roof perimeter, between the corners, with rectangular dimension and width = 0.6h from roof edges.

Zone 1: The field of roof (area inside corners and perimeter corners) up to a distance of 1.2h from the edge of the roof.

Zone 1': Applies to relatively wide buildings (>2.4h) and covers the remaining roof area inward from Zone 1.



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SHEET NO. 1 OF 1
PROJECT Auburndale
SUBJECT Wind Loads on Wall per FM Global
CLIENT Nuclei JOB NO. S99-103-26
CALCULATED BY PD DATE 5/1/2026
CHECKED BY DATE

Wind Loads on Wall per FM Global

Wind Pressure Based on FM Global:

			Reference
Velocity Pressure Coefficient, K_z =	1.04	Based on a mean roof height of 40'-10"	(FMDS0128, Table 3.2.2g)
Topographic Factor, K_{zt} =	1.0	Flat terrain	(FMDS0128, Section 3.2.1)
Directionality Factor, K_D =	0.85		(FMDS0128, Section 3.2.1)
Elevation Factor, K_E =	1.0	Elevation near sea level	(FMDS0128, Section 3.2.1)
Design Wind Speed, V =	100	mph	(FMDS0128, Appendix C Fig 3)
Importance Factor, I =	1.15		(FMDS0128, Section 3.2.1)

Wind Velocity Pressure, qh = $0.00256K_ZK_{Zt}K_DK_EV^2I$ = **26.02** psf

Wind Pressure for Specific Wall Zones

Building Mean Roof Height, h = 40.83 ft

External Pressure Coefficient, GCp =	Zone 5	Zone 4	(FMDS0128, Table 3.2.2e)
	-1.26	-0.99	
	0.9	0.9	

Internal Pressure Coefficient, GCpi = 0.18 and -0.18 Enclosed building (FMDS0128, Section 3.2.1)

Design Wind Pressure, P = $[(GC_p) - (GC_{pi})]q_h$ = (FMDS0128, EQ 3.2.1b)

Safety Factor, SF = 2.0

Nominal Ultimate Design Wind Pressure, Pu = P(SF)

		+Gcpi P, psf	-Gcpi -P, psf	+Gcpi +Pu, psf	-Gcpi -Pu, psf
-GCp	Zone 5	-37.48	-28.11	**-74.95**	**-56.21**
+GCp	Zone 5	18.74	28.11	**37.48**	**56.21**
-GCp	Zone 4	-30.45	-21.08	**-60.90**	**-42.16**
+GCp	Zone 4	18.74	28.11	**37.48**	**56.21**

Zone 5 Width, a = **16.33** ft
lesser of 10% least plan dimension or 0.4h, but not less than 4% of the least plan dimension or 3ft

Building Width = 260.0 ft
Building Length = 864.0 ft

Table 3.2.2e. Values of GC_p for Wall Zones 4 and 5

Building Height	Roof Slope	Zone 4	Zone 5
H ≤ 60 ft (18 m); or 60 ft <H <90 ft (27.4 m) for h/w ≤ 1.0	≤10°	-0.99	-1.26
		+0.9	+0.9
	>10°	-1.1	-1.4
		+1.0	+1.0
H > 90 ft (18 m) for h/w >1.0; or H >90 ft (27.4 m) for h/w ≤1.0	All slopes	-0.9	-1.8
		+0.9	+0.9

Note: The width (a) of Zone 5 is the lesser of 10% of the least plan dimension, or 0.4h, but not less than 4% of the least plan dimension or 3 ft (0.9 m)

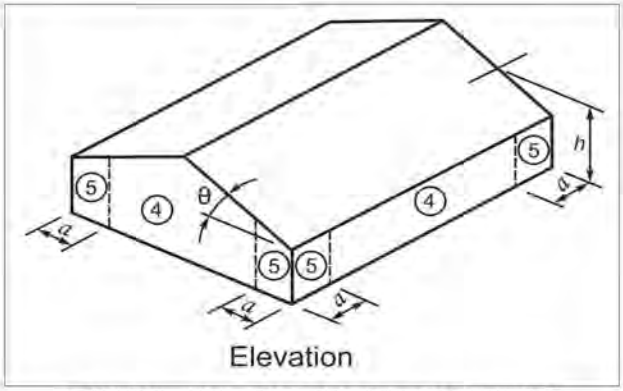


Fig. 3.2.2e. Wall zones for buildings ≤60 ft (18 m) high, or Buildings <90 ft (27 m) where h/w ≤ 1

2.0 DECK DESIGN

Summary of deck requirements and checks

Goal is adherence to FM documents 1-28 and 1-29

1. Determine wind pressures (confirmed excel sheet is accurate and matches RoofNav)

2. Check maximum deck span from FM 1-29 Table 1A (page 11 of 50 in 1-29, attached on next sheet of this file)
   Wind rating 180 (corner zones): **min. 18 ga deck required for 6' span**
   Wind rating 135 (edge zones): **min. 18 ga deck required for 6' span, otherwise provide supports at 4.5'**
   Wind rating 105 (Zones 1 and 1'): 6' max span (OK)

3. Zone 1 wind exceeds 1-90 requirement:
   Deck yield strength is required to be 80 ksi (OK)
   Deck securement per prescriptive enhanced approach, see note 4

4. Zone 1 = 6" fastener spacing <- Use engineered solution, see following pages
   Zone 2 = 4" fastener spacing <- Use engineered solution, see following pages
   Zone 3 = 3" fastener spacing <- Use engineered solution, see following pages



5. Determine sidelap fastener requirement per FM 1-29 Table 4
   Zone 1 = 30" OC <- Use engineered solution, see following pages
   Zone 2 = 15" OC <- Use engineered solution, see following pages
   Zone 3 = 15" OC <- Use engineered solution, see following pages

Table 1B. Maximum Steel Deck Span (ft) for 1½ in. (38 mm) Deep, Yield Stress ≥ 60,000 psi (414 MPa) with a mechanically fastened Roof Cover (continued)
(Note: Use this table when the distance between rows of roof cover fasteners is more than one-half the deck span.)

Max Deck Spans By Wind Rating/Fastener Spacing, Sheet Gauge for 80 ksi, 1½ in. Deep Wide Rib Deck																			
Roof Cover Fastener Row Spacing (ft)	Gauge	Wind Rating [psf]																	
		330	315	300	285	270	255	240	225	210	195	180	165	150	135	120	105	90	75	60
8.5	18	-	-	-	-	-	4	4	4.5	5	5.5	6	6	6	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	4	4	4.5	5.5	6	6	6	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	6	6	6	6
9	18	-	-	-	-	-	-	4	4	4.5	5	5.5	6	6	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	4	4.5	5	5.5	6	6	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5.5	6	6	6
9.5	18	-	-	-	-	-	-	4	4	4	4.5	5	5.5	6	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	4	4	4.5	5	6	6	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	6	6	6
10	18	-	-	-	-	-	-	-	4	4	4.5	4.5	5	6	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	-	4	4.5	4.5	5.5	6	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4	4.5	5.5	6	6
10.5	18	-	-	-	-	-	-	-	4	4	4.5	4.5	5	5.5	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	-	4	4	4.5	5	6	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5.5	6	6
11	18	-	-	-	-	-	-	-	-	4	4	4.5	5	5	6	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	5.5	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	6	6
11.5	18	-	-	-	-	-	-	-	-	-	4	4	4.5	5	5.5	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	5.5	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	6	6
12	18	-	-	-	-	-	-	-	-	-	4	4	4.5	5	5.5	6	6	6	6	6
	20	-	-	-	-	-	-	-	-	-	-	-	-	-	4	4.5	5	6	6	6
	22	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	5	5.5	6
Roof Cover Fastener Row Spacing	Gauge	330	315	300	285	270	255	240	225	210	195	180	165	150	135	120	105	90	75	60
		Wind Rating [psf]																		

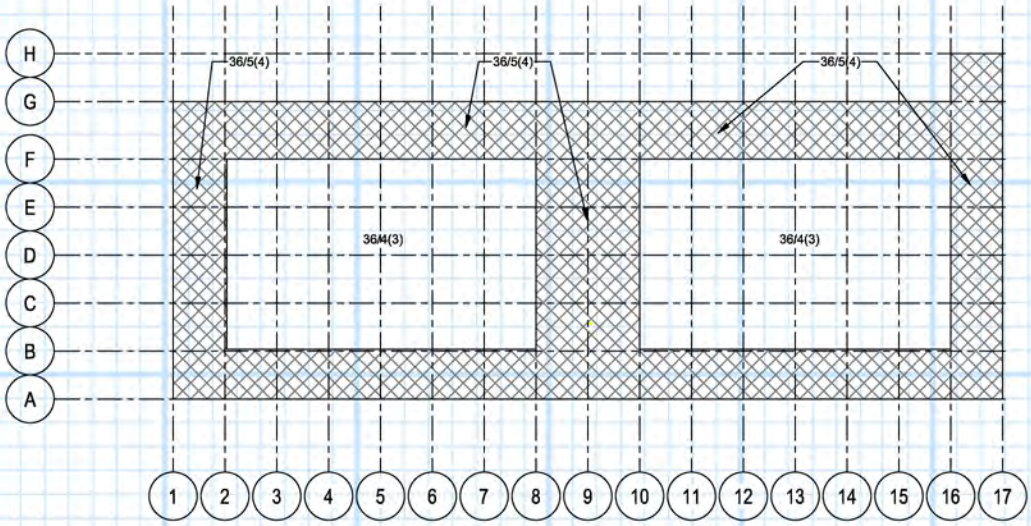
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Summary of deck fastener checks

	ORIGINAL	NO BALLAST	+10 PSF BALLAST
	ZONE 1 = -26.3 PSF	-56.4 PSF	-50.4 PSF
	ZONE 2 = -36.8 PSF	-75.9 PSF	-69.9 PSF
	ZONE 3 = -36.8 PSF	-105.2 PSF	-99.2 PSF
	ZONE 1' = -11.5 PSF	-30.4 PSF	-24.4 PSF

NOTE:
ASCE7-05 SERVICE LOAD
COMBO = 06.D+1.0W

ALTERNATE:
1. USE 36/4(2) FIELD AND 36/5(2) PERIMETER WITH HILTI-X-HSN 24 PINS AND HILTI S-SLC 02M HWH SIDE LAP CONNECTORS.



2 S-211 **OVERALL DECK CONNECTION PLAN**
SCALE: 1" = 100'-0"

Summary of deck fastener checks

No change to lateral diaphragm design - by inspection MWFRS forces control lateral diaphragm design and remain unchanged.

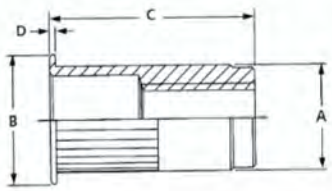
See following pages for check of existing deck with existing fasteners. Note 35/5(2) is not an option on Verco software so this layout is input as 35/9 instead (conservatively uses less fasteners.

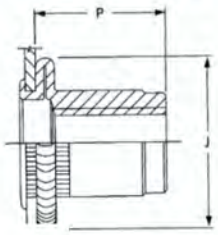
Existing fasteners OK for updated FM loading.

Zone 1 and 1':
Max wind net uplift = 56.4 psf (ASD)
For serviceability, adjust to equivalent 10 year return period per IBC, use pressure = 0.7 * 56.4 psf = 39.5 psf
Deck capacity (strength) = 127 psf > 56.4 psf OK
Deck capacity (deflection) = 46.8 psf > 39.5 psf OK

Zone 3 (worst case for full perimeter:
Max wind net uplift = 105.2 psf (ASD)
For serviceability, adjust to equivalent 10 year return period per IBC, use pressure = 0.7 * 105.2 psf = 73.6 psf
Additional deck supports added, use 3' deck span.
Deck capacity (strength) = 508 psf > 105 psf OK
Deck capacity (deflection) = 375 psf > 74 psf OK

To keep all work on underside of deck, use #10 threaded rivet nuts. See product data below. Note rivet nut diameter is larger than the shot pin diameter and the minimum grip diameter exceeds that of shot pins and #12 screws. By inspection this substitution does not effect the deck capacity for uplift or shear. Utilize rivet nuts at the same spacing as existing deck connections.





ıert After Placing
Minimum Grip)

RIVET NUTS: K-SERIES

ALUMINUM - 5056 plain finish
STEEL - C1008, yellow zinc (Bu)
STAINLESS STEEL - 18-8

Thread Designation	Grip Range	Hole +.006 Dia. -.000	Steel Series	Aluminum Series	18-8 S/S Series	A Max.	B Nom.	C Max.	P Ref.	D Max.	J Max.
#6-32 UNC	.020-.080	17/64 (.2656)	RN632080SKR	RN632080AKR	RN632080FKR	.265	.310	.450	.305	.019	.390
#6-32 UNC	.080-.130	17/64 (.2656)	RN632130SKR	RN632130AKR	RN632130FKR	.265	.310	.500	.305	.019	.390
#8-32 UNC	.020-.080	17/64 (.2656)	RN832080SKR	RN832080AKR	RN832080FKR	.265	.310	.450	.305	.019	.390
#8-32 UNC	.080-.130	17/64 (.2656)	RN832130SKR	RN832130AKR	RN832130FKR	.265	.310	.500	.305	.019	.390
#10-24 UNC	.020-.130	19/64 (.2969)	RN1024130SKR	RN1024130AKR	RN1024130FKR	.296	.340	.505	.315	.020	.450
#10-24 UNC	.130-.225	19/64 (.2969)	RN1024225SKR	RN1024225AKR	RN1024225FKR	.296	.340	.615	.315	.020	.450
#10-32 UNF	.020-.130	19/64 (.2969)	RN1032130SKR	RN1032130AKR	RN1032130FKR	.296	.340	.505	.315	.020	.450
#10-32 UNF	.130-.225	19/64 (.2969)	RN1032225SKR	RN1032225AKR	RN1032225FKR	.296	.340	.615	.315	.020	.450
1/4-20 UNC	.027-.165	25/64 (.3906)	RN2520165SKR	RN2520165AKR	RN2520165FKR	.390	.455	.610	.380	.021	.550
1/4-20 UNC	.165-.260	25/64 (.3906)	RN2520260SKR	RN2520260AKR	RN2520260FKR	.390	.455	.710	.380	.021	.550
1/4-28 UNF	.027-.165	25/64 (.3906)	RN2528165SKR	RN2528165AKR	RN2528165FKR	.390	.455	.610	.380	.021	.550
1/4-28 UNF	.165-.260	25/64 (.3906)	RN2528260SKR	RN2528260AKR	RN2528260FKR	.390	.455	.710	.380	.021	.550
5/16-18 UNC	.027-.150	17/32 (.5312)	RN5161815SKR	RN5161815AKR	RN5161815FKR	.530	.595	.720	.470	.023	.740

22 ga NSB™-36 Grade 80 Class 3

Bare Deck Inward and Outward Uniform Load

Calculated in accordance with IAPMO ER-2018, AISI S100-16 (2020) w/S2-20

VERCO DECKING, INC.

#12 Screw Connections to Supports
36 / 5 Perpendicular Connection Pattern to Supports
A572 GR50 Support Member or Equivalent
0.25 ≤ Support Thickness (in.) ≤ 0.5

End Bearing Length of 2.5 in.
Interior Bearing Length of 5 in.
L/180 Deflection Limit
Sheet lengths are limited to 42 ft

End Connection Pattern



Interior Connection Pattern



Inward Uniform LRFD Design Load Table (psf)

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	794	583	447	353	286	236	198	169	146
2	814	604	465	369	300	249	209	179	154
3	1001	746	576	458	373	309	261	223	192

Inward Uniform ASD Design Load Table (psf)

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	528	388	297	235	190	157	132	112	97
2	541	402	310	246	200	166	139	119	103
3	666	496	383	305	248	206	174	148	128

Inward Uniform SERVICE Design Load Table, L/180 Deflection Limit (psf)

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	579	364	245	172	125	94	72	57	46
2	1505	946	636	446	325	244	188	148	119
3	1179	741	498	349	254	192	147	116	93

Outward Uniform LRFD Design Load Controlled by Panel Strength Table (psf)

846 * 0.6 = 508 psf ASD

212 * 0.6 = 127 psf ASD

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	846	622	476	376	305	252	212	180	155
2	767	568	438	347	282	234	197	168	145
3	945	703	543	431	351	291	245	209	181

Outward Uniform LRFD Design Load Controlled by Connection Strength Table (psf)

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	293	251	220	195	176	160	146	135	126
2	234	201	176	156	141	128	117	108	100
3	266	228	200	178	160	145	133	123	114

Outward Uniform SERVICE Design Load Table, L/180 Deflection Limit (psf)

625 * 0.6 = 375 psf ASD

78 * 0.6 = 46.8 psf ASD

No. of Spans	Span								
	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
1	625	393	264	185	135	101	78	61	49
2	1396	878	590	414	301	227	174	137	110
3	1094	688	462	324	236	178	137	107	86

SDI Construction Span

Point Load	Uniform Load	Span Condition		
(lb)	(psf)	1 Span	2 Span	3 Span
200.0	20.0	12'-3"	14'-11"	15'-2"

Strength Properties

Property	LRFD			ASD		
Positive Bending Strength	$\phi_b \cdot M_{n+} =$	0.893	k-ft/ft	$M_{n+} / \Omega_b =$	0.594	k-ft/ft
Negative Bending Strength	$\phi_b \cdot M_{n-} =$	0.952	k-ft/ft	$M_{n-} / \Omega_b =$	0.634	k-ft/ft
Vertical Web Shear Strength	$\Phi_v \cdot V_n =$	5.555	kips/ft	$V_n / \Omega_v =$	3.654	kips/ft
Vertical Shear, Normal Position (Cell Deck only)	$\phi \cdot V_{nn} =$	-	kips/ft	$V_{nn} / \Omega =$	-	kips/ft
Vertical Shear, Inverted Position (Cell Deck only)	$\phi \cdot V_{ni} =$	-	kips/ft	$V_{ni} / \Omega =$	-	kips/ft
End Web Crippling Strength (2.5 in. bearing)	$\phi_w \cdot P_{ns1} =$	2.192	kips/ft	$P_{ns1} / \Omega_w =$	1.433	kips/ft
Interior Web Crippling Strength (5 in. bearing)	$\phi_w \cdot P_{ni1} =$	3.565	kips/ft	$P_{ni1} / \Omega_w =$	2.397	kips/ft

Version: V1.0.0 Beta

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3.0 WALL DESIGN

Summary of Tilt-Up Panel Loadings

Tilt-Up Panels:
Ref joist and deck loadings for gravity loads from roof

SERVICE LEVEL WALL WIND PRESSURES

ZONE 4 = -60.9 PSF / 1.6 = 38.1 PSF

ZONE 5 = -74.95 PSF / 1.6 = 46.9 PSF

Gravity loads from roof:
Trib width = 55'

DL (NO BALLAST) = 13 psf * 55/2 = 358 plf

DL (+10psf BALLAST) = 23 psf * 55/2 = 633 plf

Lr = 20psf * 55/2 = 550 plf

W_gravity = 16.7 psf * 55/2 = 460 plf

GRAVITY LOADS:

SLAB ON GRADE LIVE LOAD	250 PSF
ROOF DEAD LOAD	
ROOF MEMBRANE AND INSULATION	2 PSF
METAL DECK	2 PSF
JOIST FRAMING	3 PSF
SPRINKLERS	3 PSF
DUCTS, LIGHTING, MISC	3 PSF
TOTAL ROOF DEAD LOAD	13 PSF
ROOF LIVE LOAD (REDUCIBLE)	20 PSF
DOWNWARD (POSITIVE) C&C WIND LOAD	16.7 PSF (ULTIMATE)

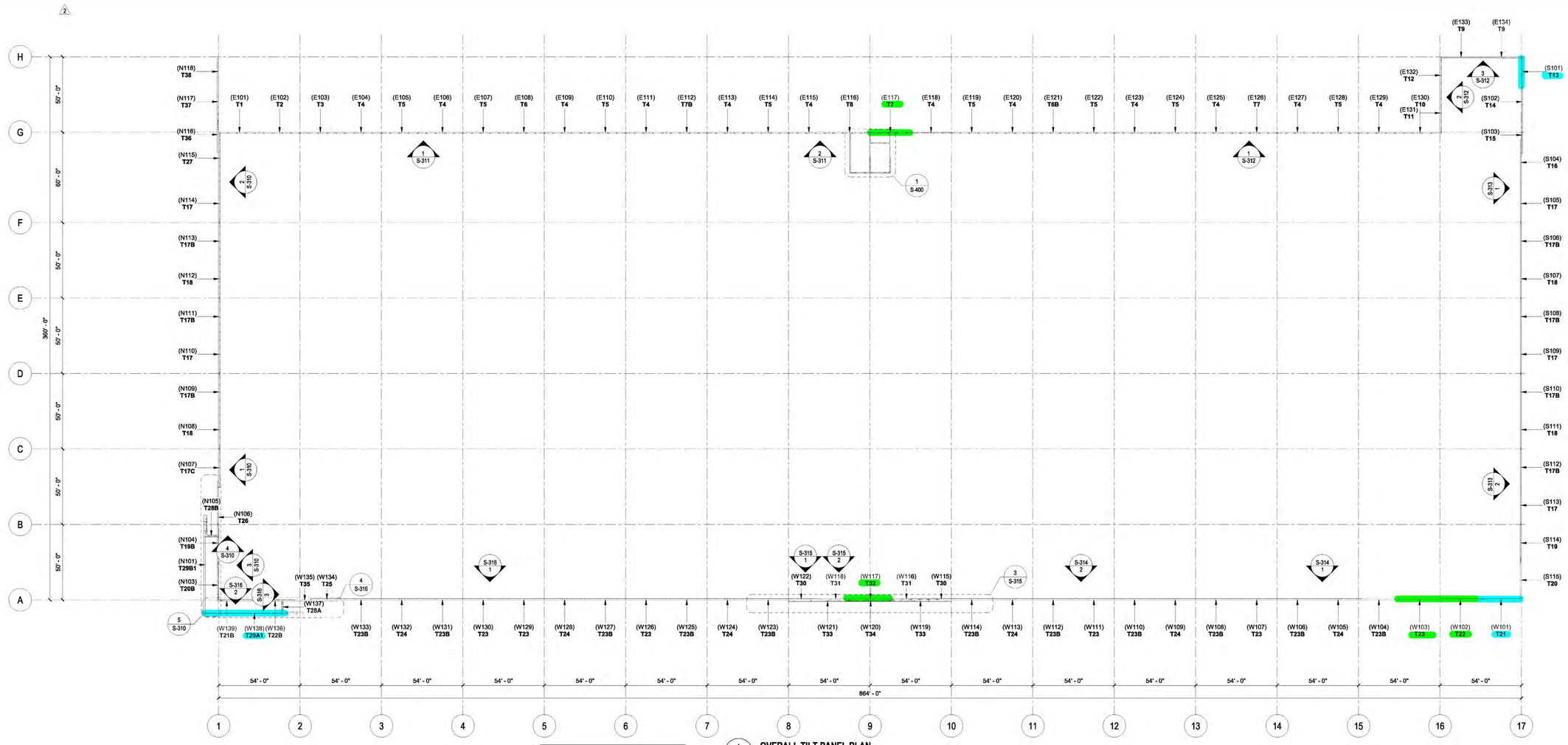
JOIST LOAD NOTES:
1. ALL LOADS SHOWN ARE UNFACTORED
2. ALL CODE LOAD COMBINATIONS AND LIMIT STATES SHOULD BE ACCOUNTED FOR BY JOIST DESIGNER

WALL PRESURES

		+Gcpi P, psf	-Gcpi -P, psf	+Gcpi +Pu, psf	-Gcpi -Pu, psf
-GCp	Zone 5	-37.48	-28.11	-74.95	-56.21
+GCp	Zone 5	18.74	28.11	37.48	-56.21
-GCp	Zone 4	-30.45	-21.08	-60.90	-42.16
+GCp	Zone 4	18.74	28.11	37.48	-56.21

Zone 5 Width, a = 16.33 ft

ULTIMATE WIND PRESSURES. DIVIDE BY 1.6 FOR SERVICE LEVEL



DENOTES PANELS WITH ZONE 5 LOADING (WORST CASE WIND)

ENVELOPE worst cases, Panels to check:
T29A1/T29A2 (overhang panel)
T21 (corner with openings)
T13 (corner, no openings)

DENOTES PANELS WITH ZONE 4 LOADING

ENVELOPE worst cases, Panels to check:
T7 (interior, lots of openings)
T22 (3 stacked openings)
T23 (solid wall)
T30 (solid wall w/ #6 vertical bars)

TILT PANEL SCHEDULE

T1	TH = 9 1/4"	F'c = 4000 PSI
T2	TH = 9 1/4"	F'c = 4000 PSI
T3	TH = 9 1/4"	F'c = 4000 PSI
T4	TH = 9 1/4"	F'c = 4000 PSI
T5	TH = 9 1/4"	F'c = 4000 PSI
T6	TH = 9 1/4"	F'c = 4000 PSI
T6B	TH = 9 1/4"	F'c = 4000 PSI
T7	TH = 9 1/4"	F'c = 4000 PSI
T7B	TH = 9 1/4"	F'c = 4000 PSI
T8	TH = 9 1/4"	F'c = 4000 PSI
T9	TH = 9 1/4"	F'c = 4000 PSI
T10	TH = 9 1/4"	F'c = 6000 PSI
T11	TH = 9 1/4"	F'c = 4000 PSI
T12	TH = 9 1/4"	F'c = 4000 PSI
T13	TH = 9 1/4"	F'c = 4000 PSI
T14	TH = 9 1/4"	F'c = 4000 PSI
T15	TH = 9 1/4"	F'c = 4000 PSI
T16	TH = 9 1/4"	F'c = 4000 PSI
T17	TH = 9 1/4"	F'c = 4000 PSI
T17B	TH = 9 1/4"	F'c = 4000 PSI
T17C	TH = 9 1/4"	F'c = 4000 PSI
T18	TH = 9 1/4"	F'c = 4000 PSI
T19	TH = 11 1/4"	F'c = 6000 PSI
T19B	TH = 11 1/4"	F'c = 6000 PSI
T20	TH = 11 1/4"	F'c = 6000 PSI
T20B	TH = 11 1/4"	F'c = 6000 PSI
T21	TH = 11 1/4"	F'c = 6000 PSI
T21B	TH = 11 1/4"	F'c = 4000 PSI
T22	TH = 11 1/4"	F'c = 6000 PSI
T22B	TH = 11 1/4"	F'c = 4000 PSI
T23	TH = 9 1/4"	F'c = 4000 PSI
T23B	TH = 9 1/4"	F'c = 4000 PSI
T24	TH = 9 1/4"	F'c = 4000 PSI
T25	TH = 9 1/4"	F'c = 4000 PSI
T26	TH = 9 1/4"	F'c = 4000 PSI
T27	TH = 9 1/4"	F'c = 6000 PSI
T28A	TH = 12"	F'c = 4000 PSI
T28B	TH = 12"	F'c = 4000 PSI
T29A1	TH = 24"	F'c = 4000 PSI
T29A2	TH = 12"	F'c = 4000 PSI
T29B1	TH = 24"	F'c = 4000 PSI
T29B2	TH = 12"	F'c = 4000 PSI
T30	TH = 9 1/4"	F'c = 4000 PSI
T31	TH = 10 1/2"	F'c = 6000 PSI
T32	TH = 10 1/2"	F'c = 6000 PSI
T33	TH = 8"	F'c = 6000 PSI
T34	TH = 8"	F'c = 6000 PSI
T35	TH = 9 1/4"	F'c = 4000 PSI
T36	TH = 11 1/4"	F'c = 4000 PSI
T37	TH = 11 1/4"	F'c = 4000 PSI
T38	TH = 11 1/4"	F'c = 4000 PSI
TPRA	TH = 7 1/4"	F'c = 4000 PSI
TPRB	TH = 7 1/4"	F'c = 4000 PSI
TPRC	TH = 7 1/4"	F'c = 4000 PSI
TPRD	TH = 7 1/4"	F'c = 4000 PSI
TPRE	TH = 7 1/4"	F'c = 4000 PSI

1 S-300 OVERALL TILT PANEL PLAN

SCALE: 1" = 30'-0"

TILT PANEL PLAN NOTES:

1. TILT PANELS ARE DRAWN IN ELEVATION FROM THE INSIDE OF THE BUILDING.
2. TILT PANELS SHOULD REMAIN BRACED UNTIL THE ENTIRETY OF THE ROOF DECK AND ROOF FRAMING HAS BEEN CONNECTED TO THE PANELS, AND THE CONCRETE SLAB POUR STRIP CONCRETE HAS BEEN PLACED. IN LIEU OF POUR STRIP CONCRETE MECHANICAL CONNECTIONS CAN BE MADE FROM THE BASE OF THE TILT PANEL TO THE FOOTING.
3. REFER TO THE ARCHITECTURAL DRAWINGS FOR CONFIRMATION OF ALL PANEL DIMENSIONS, AND ALL REVEALS.
4. SEE TILT PANEL ELEVATIONS FOR PANEL THICKNESSES.
5. SEE TILT PANEL REBAR ELEVATIONS FOR PANEL REINFORCEMENT.
6. ALL REINFORCEMENT TO BE 5A FACE UNLESS NOTED OTHERWISE.
7. TYPICAL REINFORCING UNLESS NOTED OTHERWISE ON PANEL REBAR ELEVATIONS:
   A. #5 + HK, EF AT TOP OF PANEL.
   B. (2) #5 AT 8" OC +HK EF @ BOTTOM OF PANEL.
   C. #5 + HK, EF AT TOP OF OPENINGS.
   D. #4 DOWELS AT 18 INCHES OC FOR DOWELS , TYPICAL. PROVIDE MINIMUM OF (3) DOWELS PER LEG.
   E. TIES #3 AT 7 INCHES OC FOR VERTICAL BARS AT PANEL LEGS ADJACENT TO OPENINGS OR FUTURE OPENINGS.
8. SEE DETAILS ON S-520 FOR DETAILS AT GIRDER BEARING.
9. TILT PANEL SHALL BE NORMAL WEIGHT WITH f'_c = 4000 PSI MINIMUM AT 28 DAYS, UNLESS NOTED OTHERWISE.
10. REINFORCEMENT SHALL BE CONTINUOUS THROUGH KNOCKOUT AREA (FUTURE OPENING).
11. TRIM VERTICAL BARS BACK AT STEPPED EDGES, TYPICAL TOP OF TILT PANEL.
12. SEE PANEL ELEVATIONS FOR LOCATIONS OF PANEL EMBED PLATES.
13. ALL TILT PANELS ARE ASSUMED TO BE PAINTED AND/OR WATERPROOFED, SEE ARCHITECTURAL DRAWINGS. UNPAINTED/UNPROTECTED PANELS SHALL HAVE 3% AIR ENTRAINMENT. CONCRETE SHALL BE A MINIMUM OF 3000 PSI AT THE TIME OF ERECTION. (CONCRETE SLUMP SHALL NOT EXCEED 4 INCHES).
14. FLY ASH AND/OR SLAG SHALL NOT BE PERMITTED IN TILT PANELS.
15. TILT PANELS CONSTRUCTION IS TO CONFORM TO ACI 117-90. (STANDARD TOLERANCES FOR CONCRETE CONSTRUCTION AND MATERIALS).
16. REINFORCING STEEL SHALL CONFORM TO ASTM A615 GRADE 60. REINFORCING STEEL DETAILING, FABRICATION AND PLACEMENT SHALL CONFORM TO THE LATEST EDITION OF CRSI'S (MANUAL OF STANDARD PRACTICE). REINFORCING STEEL IS NOT TO BE SPLICED WITHOUT THE WRITTEN APPROVAL FROM THE STRUCTURAL ENGINEER.
17. SUBMIT SHOP DRAWINGS SHOWING ALL WORK RELATED TO TILT PANELS, INCLUDING TEMPORARY BRACING AND ERECTION PROCEDURES. REINFORCING BARS SHOWN ON THE CONTRACT DRAWINGS DO NOT ALLOW FOR LIFTING AND ERECTION STRESSES. WITH THE SHOP DRAWINGS SUBMIT DESCRIPTION COVERING ERECTION PROCEDURES, ADDITIONAL REINFORCEMENT, LIFTING HARDWARE, AND BRACING.
18. PANELS HAVE BEEN DESIGNED FOR IN PLACE STRESSES ONLY. THE GENERAL CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY BRACING, LIFTING HARDWARE, ADDITIONAL REINFORCING REQUIRED FOR LIFTING AND ERECTION PROCEDURES. REINFORCING BARS SHOWN ON CONTRACT DOCUMENTS DO NOT ALLOW FOR LIFTING AND ERECTION STRESSES.
19. PROVIDE CONTINUOUS NON-SHRINK GROUT UNDER ALL PANELS. CONTRACTOR TO GROUT UNDER ALL PANELS IMMEDIATELY AFTER PANELS ARE SET.
20. SHIM SHALL BE PLACED AT 5 FEET MAXIMUM (OR CLOSER) UNDER THE PANELS TO AVOID A CONTINUOUS FOOTING FROM BOWING UP DUE TO INVERSE SOIL PRESSURE.
21. KO INDICATES KNOCK OUT PANEL.
22. #3 LATERAL TIES SHALL BE BENT FROM STRAIGHT BARS (COIL RODS SHALL NOT BE PERMITTED) TO PREVENT BOWING OF TIES AND DECREASING CLEAR COVER.
23. SUBMIT SHOP DRAWINGS SHOWING ALL PANEL EMBED PLATES AND LOCATIONS. GENERAL CONTRACTOR SHALL COORDINATE.
24. SEE ARCHITECTURAL DRAWINGS AND DETAILS FOR PANEL JOINT AND REVEAL JOINT DETAILS NOT SHOWN ON STRUCTURAL DRAWINGS.
25. PANELS ARE TO BE CAST OUTSIDE FACE DOWN. CASTING SLAB SHALL BE CURED. SAW CUTS, CRACKS AND JOINTS IN THE CASTING SLAB SHALL BE FILLED AND LEVELED WITH A SEALANT SO AS TO MINIMIZE TRANSFER OF THE JOINT LINE TO THE PANEL FACE.
26. TILT PANEL CONTRACTOR SHALL BE RESPONSIBLE FOR THE COMPATIBILITY OF CURING AGENTS, SEALANTS AND RELEASING AGENTS UTILIZED IN THE WORK. IF PANELS ARE TO BE STACKED, THE TROWELED SURFACE SHALL BE CONSIDERED THE CASTING SLAB AND TREATED AS SUCH BONDBREAKER SHALL BE APPLIED IN ACCORDANCE WITH THE MANUFACTURERS REQUIREMENTS.
27. SEE PLAN FOR TILT PANEL THICKNESS.
28. CRACKS IN PANELS DUE TO LIFTING OR SHRINKAGE ARE TO BE REPAIRED BY THE GENERAL CONTRACTOR. PANELS THAT HAVE YIELDED AND CRACKED DURING LIFTING ARE TO BE REPLACED BY THE GENERAL CONTRACTOR.
29. GENERAL CONTRACTOR IS NOT TO DAMAGE FLOOR SLAB DURING PANEL ERECTION DUE TO CRANE LOADING. CONTRACTOR TO SUPPLY DESIGN ENGINEER WITH MAXIMUM CRANE LOADING ON SLAB FOR FLOOR STRENGTH EVALUATION PRIOR TO PANEL ERECTION. ANY DAMAGE TO THE FLOOR SLAB INCLUDING STRESS CRACKS RESULTING FROM LOADS EXCEEDING THOSE PROVIDED TO THE STRUCTURAL ENGINEER ARE TO BE REPAIRED BY THE GENERAL CONTRACTOR. REPLACEMENT OF SLAB WILL BE REQUIRED IN AREAS SEVERELY DAMAGED DURING LIFTING.
30. ALL TEMPORARY BRACING SHALL REMAIN IN PLACE UNTIL ROOF DECK IS FULLY INSTALLED AND INSPECTED, AND CONCRETE POUR BACK STRIP IS FULLY INSTALLED AND HAS REACHED A MINIMUM STRENGTH OF 3000 PSI.

N

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Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

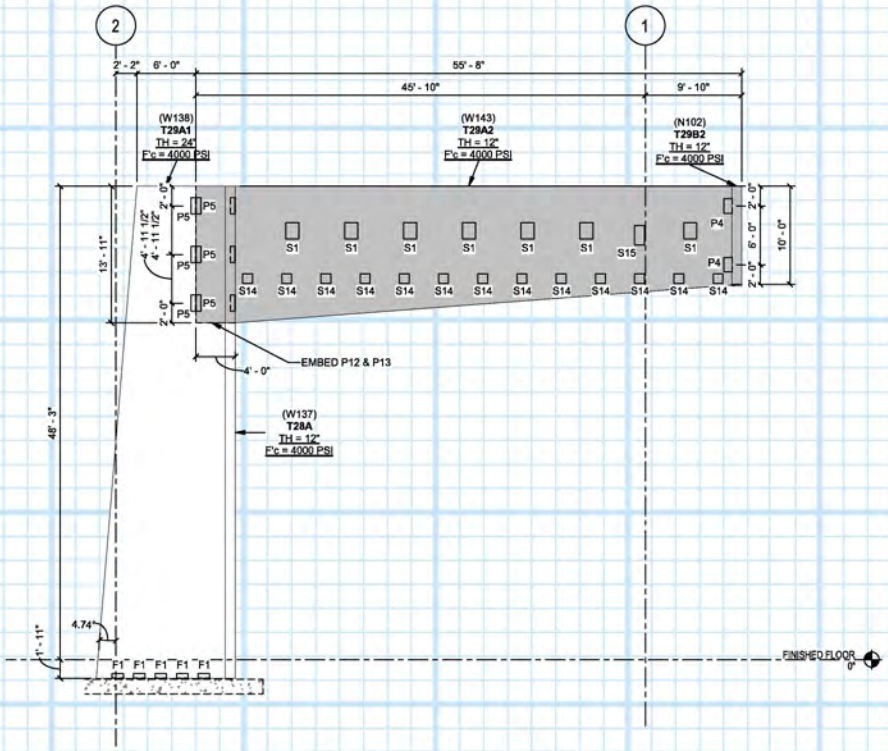
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OVERALL TILT PANEL PLAN

Sheet No.
S-300

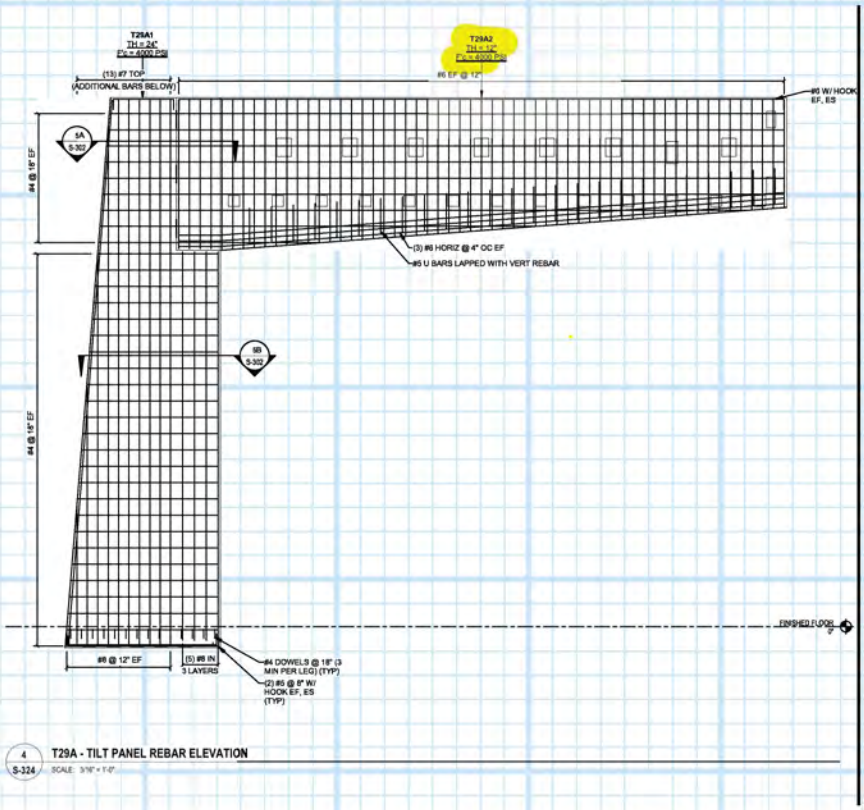
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9.17

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2 S-316 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



4 S-324 **T29A - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

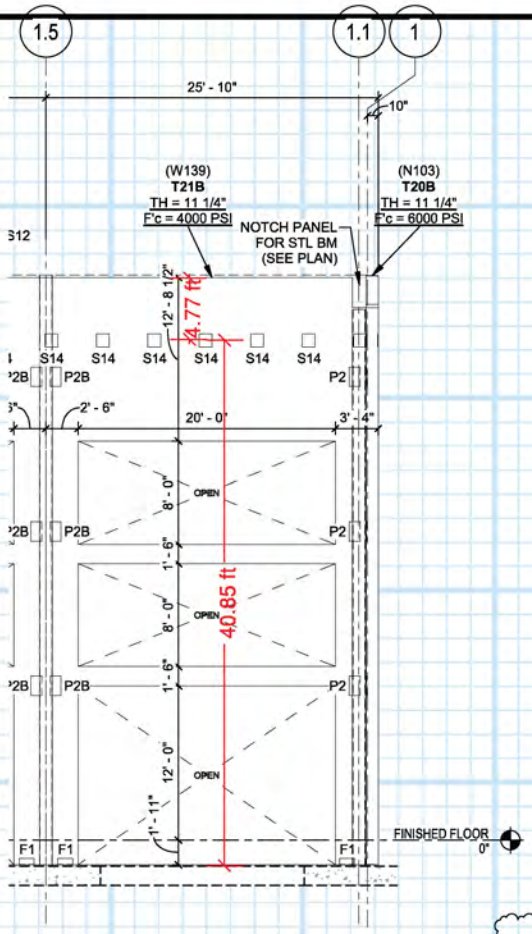
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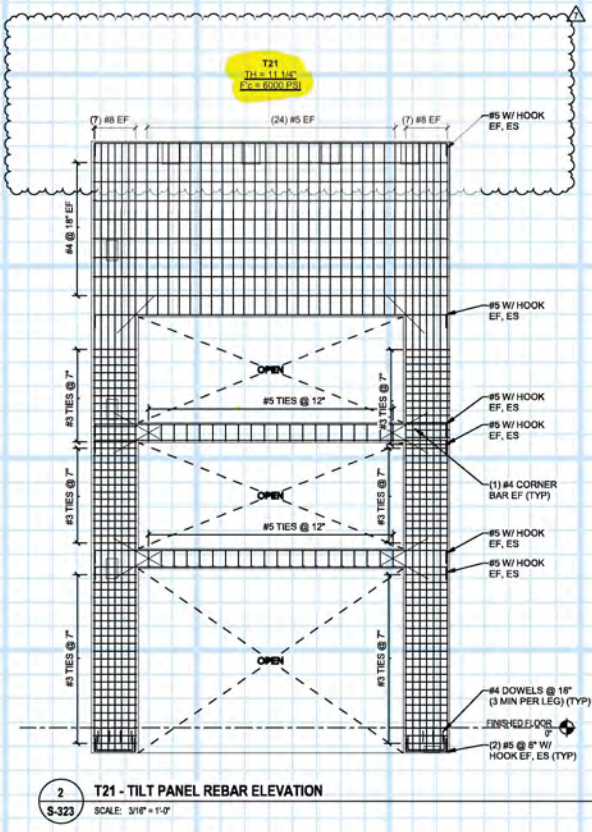
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Job No. A26-133
Project PROJECT SUNSHINE

Sheet ____ of ____
Client ____
Date 07/15/2026
Engineer BAC

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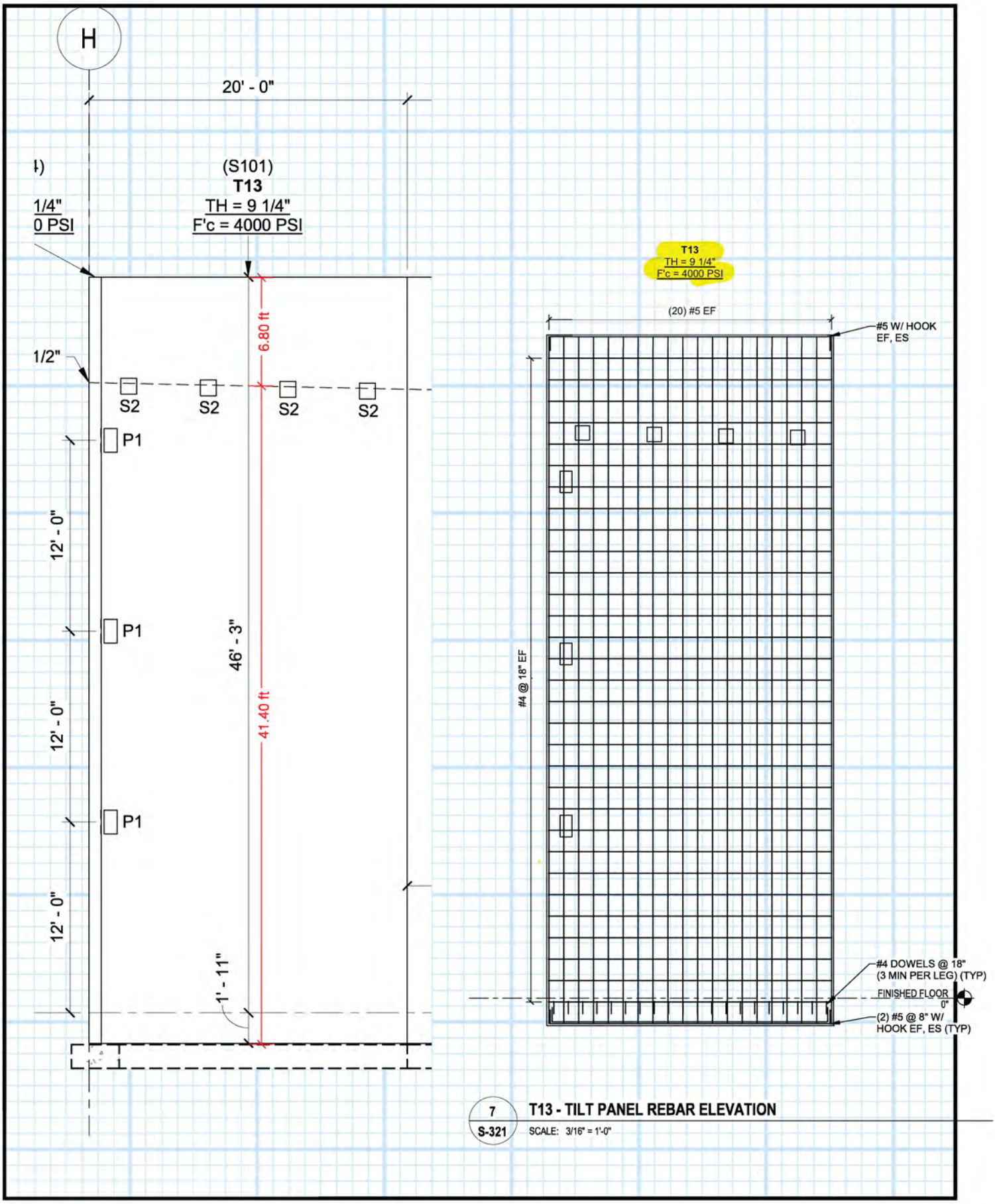
2 S-323 **T21 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Job No. A26-133 Sheet ____ of ____
Project PROJECT SUNSHINE Client ____
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7 / S-321 **T13 - TILT PANEL REBAR ELEVATION**

SCALE: 3/16" = 1'-0"

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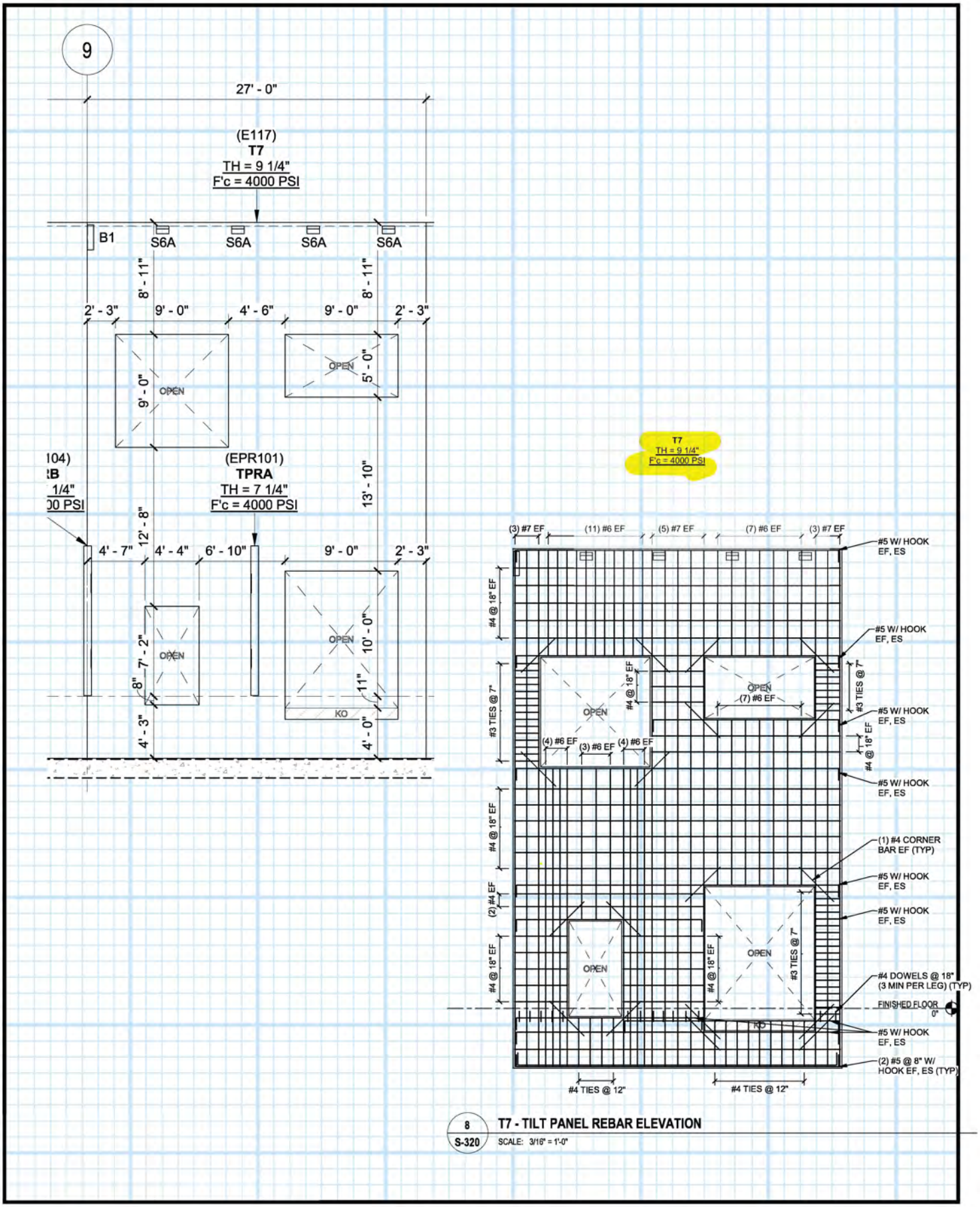
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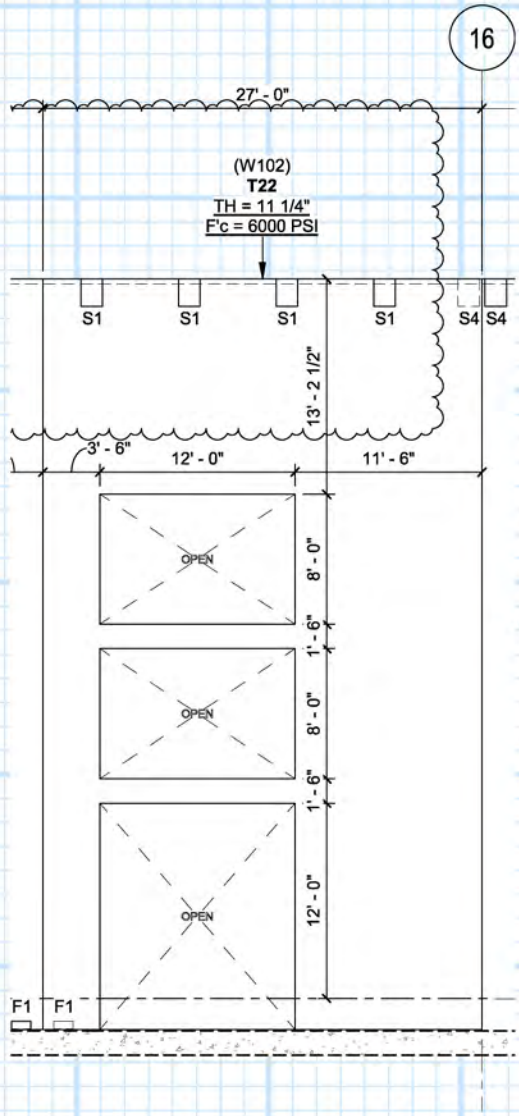
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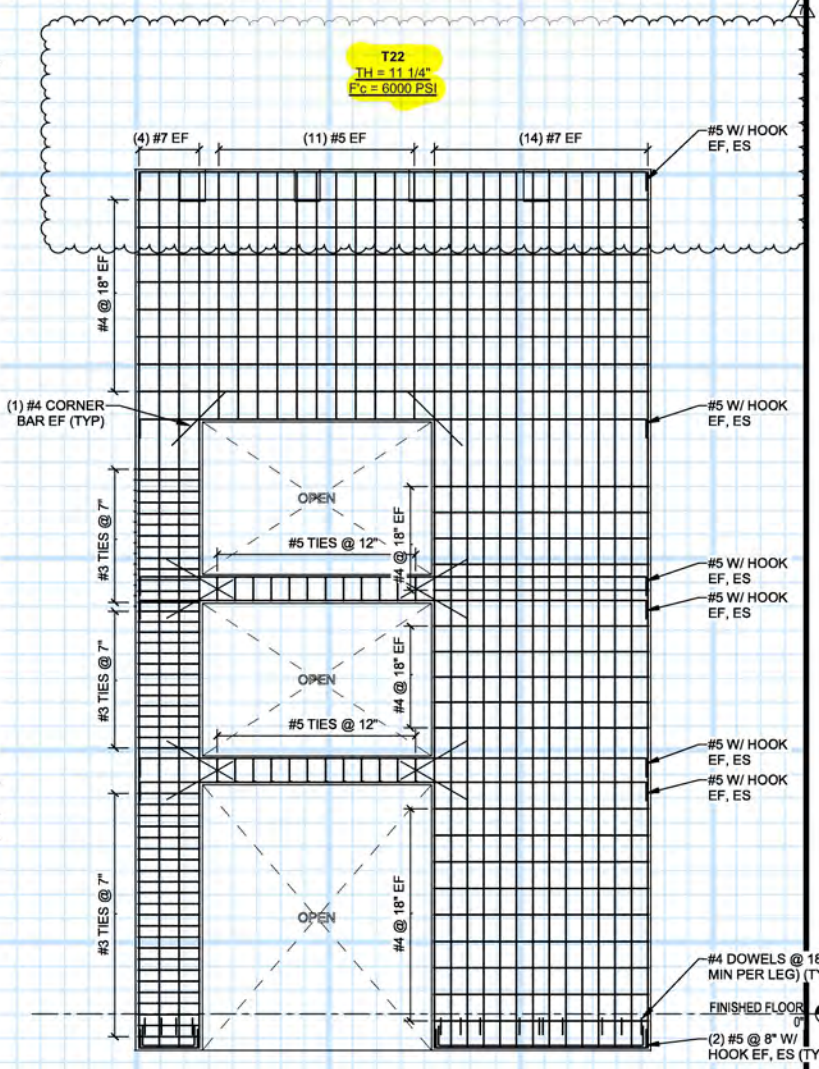
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3 S-323 **T22 - TILT PANEL REBAR ELEVATION**

SCALE: 3/16" = 1'-0"

AXIOM

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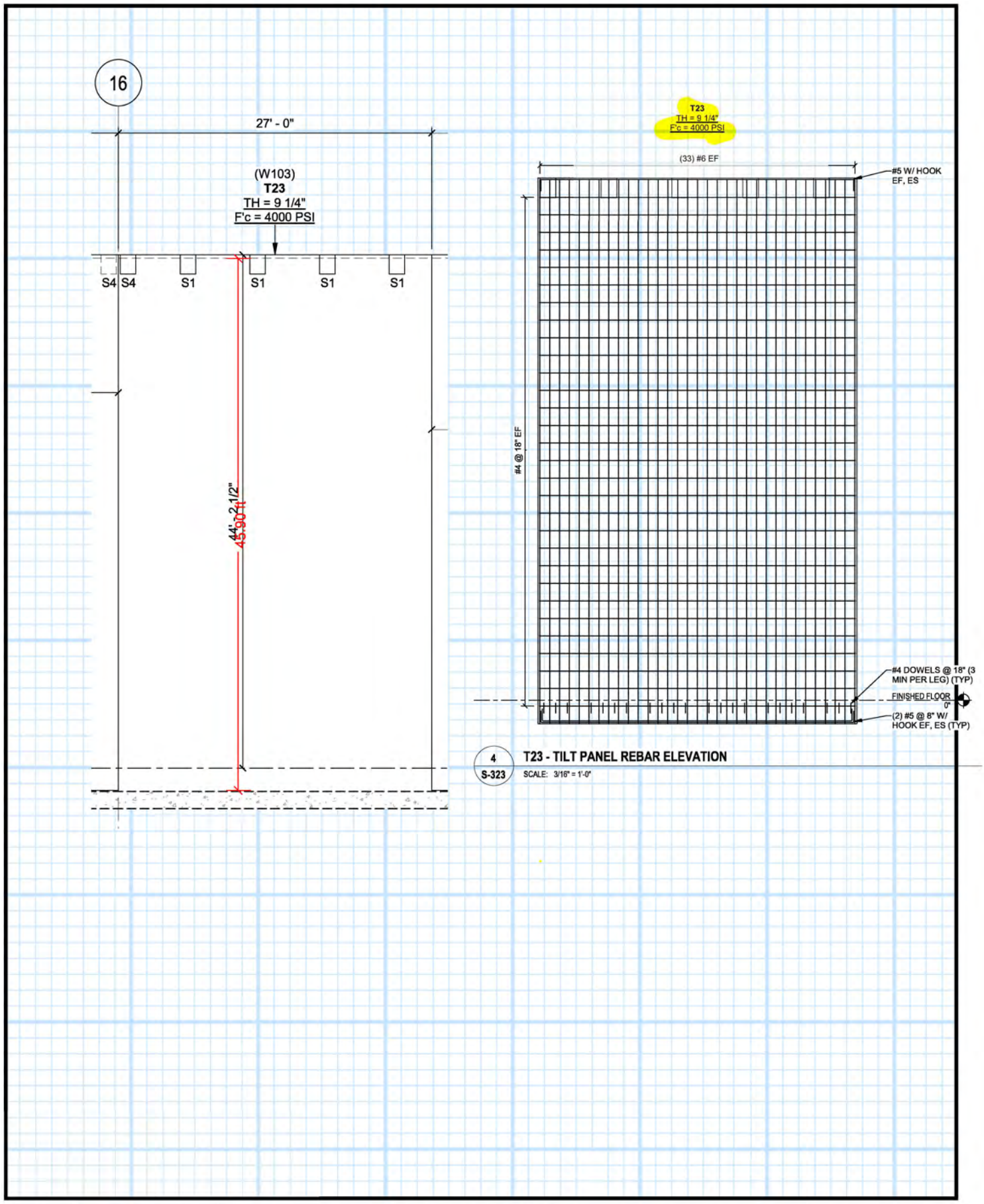
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Project PROJECT SUNSHINE Client ____

Date 07/15/2026

Engineer BAC

999 W Main St, Suite 201 • Boise, ID 83702 • Office 208-639-4520



AXIOM

Boise, ID • Salt Lake City, UT

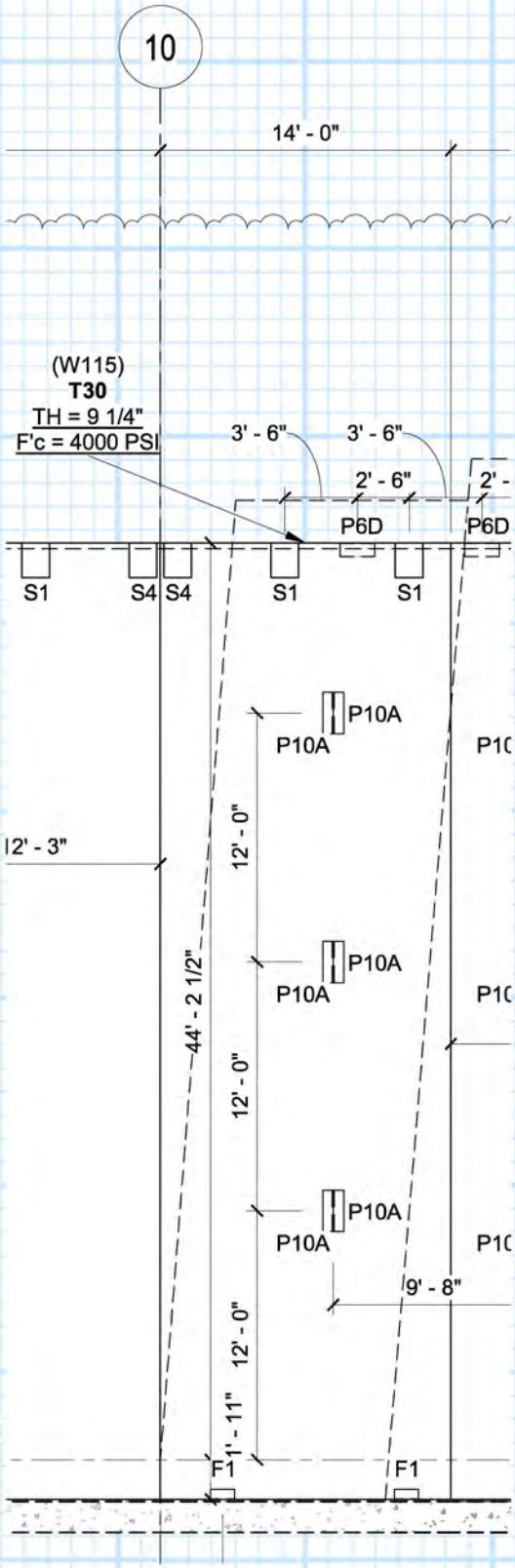
Job No. A26-133 Sheet ____ of ____

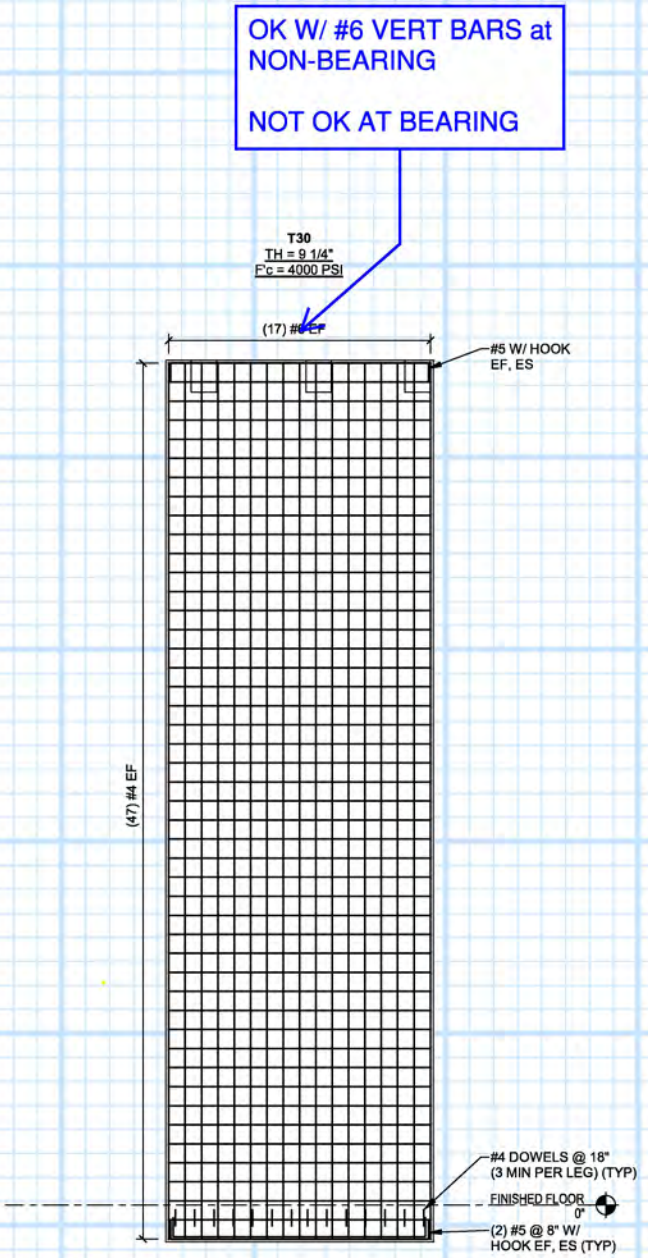
Project PROJECT SUNSHINE Client ____

Date 07/15/2026

Engineer BAC

999 W Main St, Suite 201 • Boise, ID 83702 • Office 208-639-4520





6 S-324 **T30 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

AXIOM

Boise, ID • Salt Lake City, UT

Job No. A26-133 Sheet ____ of ____
Project PROJECT SUNSHINE Client ____
Date 07/15/2026
Engineer BAC

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The following Tilt-Up Panels work for the incrfeased FM wind loading and do not require retrofit.

RAM Elements

Current Date: 8/14/2026 10:16 AM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T22_zone 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : Warnings in design

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	45.50 [ft]
Total length	:	25.83 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	45.00	11.25

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	2.50	0.00	12.00	13.92
Lower left	2.50	15.42	12.00	8.00
Lower left	2.50	24.92	12.00	8.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx

D5	Yes	1.2DL+LL+1.6Wx
D6	Yes	0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.55	0.67
1	Wx	Vertical	0.46	0.67
1	DL	Vertical	0.63	0.67

Out-of-plane pressure loads:

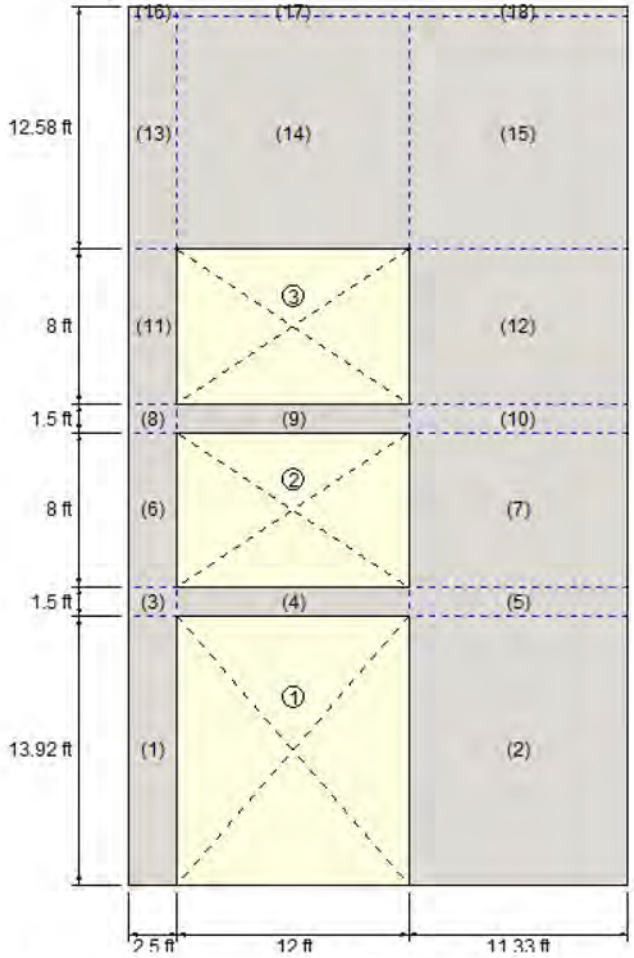
Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.50	13.92
2	14.50	0.00	11.33	13.92
3	0.00	13.92	2.50	1.50
4	2.50	13.92	12.00	1.50
5	14.50	13.92	11.33	1.50
6	0.00	15.42	2.50	8.00
7	14.50	15.42	11.33	8.00
8	0.00	23.42	2.50	1.50
9	2.50	23.42	12.00	1.50
10	14.50	23.42	11.33	1.50
11	0.00	24.92	2.50	8.00
12	14.50	24.92	11.33	8.00
13	0.00	32.92	2.50	12.08
14	2.50	32.92	12.00	12.08
15	14.50	32.92	11.33	12.08
16	0.00	45.00	2.50	0.50
17	2.50	45.00	12.00	0.50
18	14.50	45.00	11.33	0.50

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	4-#7	9.00	33.89
2	14-#7	10.00	33.89
3	4-#7	9.00	33.89
4	13-#5	11.00	19.36
5	14-#7	10.00	33.89
6	4-#7	9.00	33.89
7	14-#7	10.00	33.89
8	4-#7	9.00	33.89
9	13-#5	11.00	19.36
10	14-#7	10.00	33.89
11	4-#7	9.00	33.89
12	14-#7	10.00	33.89
13	4-#7	9.00	33.89
14	11-#5	14.00	19.36
15	14-#7	10.00	33.89
16	4-#7	9.00	33.89
17	11-#5	14.00	19.36
18	14-#7	10.00	33.89

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	49.638	0.147	0.360	0.41
2	D5 (Max)	108.381	0.071	0.360	0.20
3	D5 (Max)	46.946	0.139	0.360	0.39
4	D5 (Max)	1.774	0.001	0.360	0.00
5	D5 (Max)	104.285	0.068	0.360	0.19
6	D5 (Max)	44.165	0.131	0.360	0.36
7	D5 (Max)	88.670	0.058	0.360	0.16
8	D5 (Max)	41.805	0.124	0.360	0.34
9	D5 (Max)	1.222	0.001	0.360	0.00
10	D5 (Max)	86.689	0.057	0.360	0.16
11	D5 (Max)	41.767	0.124	0.360	0.34
12	D5 (Max)	84.499	0.055	0.360	0.15
13	D5 (Max)	20.932	0.062	0.360	0.17
14	D5 (Max)	13.445	0.008	0.360	0.02
15	D5 (Max)	56.945	0.037	0.360	0.10
16	D5 (Max)	0.207	0.001	0.360	0.00
17	D5 (Max)	0.994	0.001	0.360	0.00
18	D5 (Max)	0.939	0.001	0.360	0.00

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	2.88	1.30	1.74	9.69	0.90	199.06
2	D5 (Max)	9.45	0.88	1.18	9.69	0.90	839.19
3	D5 (Top)	2.87	1.25	1.66	9.44	0.90	187.29
4	D6 (Max)	4.04	0.33	0.44	9.56	0.90	2483.55
5	D5 (Top)	9.44	0.88	1.17	9.44	0.90	772.21
6	D5 (Top)	2.83	1.27	1.70	9.69	0.90	190.41
7	D5 (Max)	9.26	0.85	1.14	9.69	0.90	762.65
8	D5 (Top)	2.82	1.21	1.62	9.44	0.90	180.95
9	D6 (Max)	4.04	0.33	0.44	9.56	0.90	2483.55
10	D5 (Bottom)	9.27	0.85	1.14	9.44	0.90	723.09
11	D5 (Max)	2.80	1.26	1.68	9.69	0.90	189.18
12	D5 (Bottom)	9.22	0.85	1.13	9.69	0.90	761.04
13	D5 (Max)	2.61	1.08	1.44	9.56	0.90	180.66
14	D5 (Max)	3.54	0.30	0.40	9.69	0.90	2483.55
15	D5 (Max)	8.96	0.81	1.08	9.56	0.90	776.36
16	D6 (Max)	2.40	0.94	1.26	9.56	0.90	4190992.00
17	D6 (Max)	3.42	0.28	0.37	9.69	0.90	2.011676E07

18	D6 (Max)	8.41	0.73	0.97	9.56	0.90	1.899911E07

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	3559.57	1248.59	1369.46
2	D5 (Max)	16136.67	4568.81	5773.35
3	D5 (Top)	3559.57	1183.72	1288.48
4	D6 (Max)	17085.94	2213.06	17085.94
5	D5 (Top)	16136.67	4309.41	5312.53
6	D5 (Top)	3559.57	1234.86	1309.97
7	D5 (Max)	16136.67	4510.46	5246.74
8	D5 (Top)	3559.57	1172.75	1244.87
9	D6 (Max)	17085.94	2211.27	17085.94
10	D5 (Bottom)	16136.67	4261.63	4974.61
11	D5 (Max)	3559.57	1227.83	1301.49
12	D5 (Bottom)	16136.67	4497.98	5235.67
13	D5 (Max)	3559.57	1159.38	1242.84
14	D5 (Max)	17085.94	2010.44	17085.94
15	D5 (Max)	16136.67	4292.21	5341.05
16	D6 (Max)	3559.57	1108.11	3559.57
17	D6 (Max)	17085.94	1949.89	17085.94
18	D6 (Max)	16136.67	4118.03	16136.67

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-5.768
2	D6 (Max)	-3.718
3	D6 (Max)	-6.547
4	D6 (Max)	-0.001
5	D6 (Max)	-4.322
6	D6 (Max)	-7.255
7	D6 (Max)	-4.889
8	D6 (Max)	-7.753
9	D6 (Max)	-0.001
10	D6 (Max)	-5.182
11	D6 (Max)	-7.220
12	D6 (Max)	-4.851
13	D6 (Max)	-6.143
14	D6 (Max)	-0.121
15	D6 (Max)	-3.795
16	D6 (Max)	0.000
17	D6 (Max)	0.000
18	D6 (Max)	0.000

Combined axial flexure

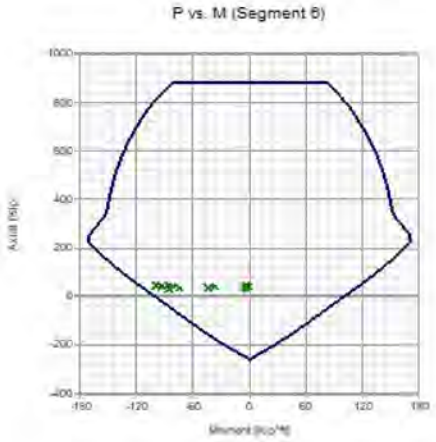
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	49.64	-55.39	-82.98	118.15	0.70
2	D5 (Max)	108.38	-171.62	-207.33	400.17	0.52
3	D5 (Top)	46.95	-58.52	-87.90	114.81	0.77
4	D6 (Max)	1.25	-0.21	-0.21	192.22	0.00
5	D5 (Top)	104.29	-182.42	-222.48	394.16	0.56
6	D5 (Top)	44.17	-68.93	-99.79	116.21	0.86
7	D5 (Max)	88.67	-209.79	-248.28	393.07	0.63
8	D5 (Top)	41.80	-69.48	-100.41	113.05	0.89
9	D6 (Max)	0.86	-0.21	-0.21	192.05	0.00
10	D5 (Bottom)	87.63	-209.80	-250.23	388.42	0.64

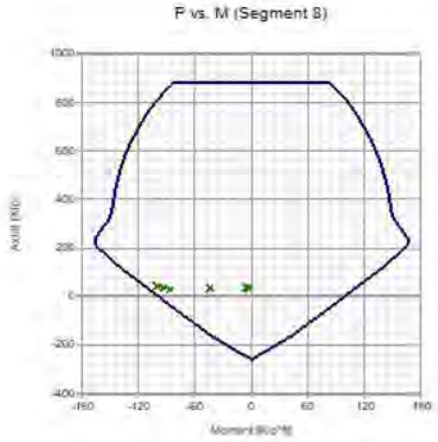
11	D5 (Max)	41.77	-69.48	-98.47	115.35	0.85
12	D5 (Bottom)	84.50	-209.48	-245.89	391.57	0.63
13	D5 (Max)	20.93	-64.72	-76.54	106.78	0.72
14	D5 (Max)	13.44	-22.72	-22.88	168.47	0.14
15	D5 (Max)	56.94	-179.00	-198.41	379.51	0.52
16	D6 (Max)	0.16	0.02	0.02	99.55	0.00
17	D6 (Max)	0.75	0.09	0.09	163.28	0.00
18	D6 (Max)	0.70	0.08	0.08	359.78	0.00

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D6 (Top)	35.13	-54.03	30.64	112.97	0.27
2	D6 (Max)	77.48	-169.05	138.88	389.05	0.36
3	D6 (Max)	33.16	-57.04	30.64	110.09	0.28
4	DC1 (Top)	-0.18	0.00	147.05	191.61	0.77
5	D6 (Max)	74.43	-179.53	138.88	383.87	0.36
6	D6 (Max)	31.01	-66.73	30.64	111.50	0.27
7	D6 (Max)	62.71	-205.36	138.88	383.73	0.36
8	D6 (Top)	29.28	-67.14	30.64	108.76	0.28
9	D1 (Max)	-0.69	0.00	147.05	191.40	0.77
10	D6 (Top)	59.82	-204.79	138.88	378.84	0.37
11	D6 (Top)	26.70	-61.60	30.64	109.96	0.28
12	D6 (Top)	48.14	-172.98	138.88	378.48	0.37
13	D6 (Top)	10.74	-2.21	30.64	103.23	0.30
14	DC1 (Top)	-4.50	-7.08	147.05	161.14	0.91
15	DC1 (Top)	19.03	-6.70	138.88	366.21	0.38
16	D6 (Top)	0.00	0.00	30.64	99.50	0.31
17	D6 (Top)	0.01	0.00	147.05	162.98	0.90
18	D6 (Top)	0.01	0.00	138.88	359.54	0.39

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	55.49	882.32	0.06
2	D5 (Bottom)	134.93	4012.99	0.03
3	D5 (Bottom)	47.57	882.32	0.05
4	D1 (Bottom)	3.32	4274.86	0.00
5	D5 (Bottom)	107.12	4012.99	0.03
6	D5 (Bottom)	47.53	882.32	0.05
7	D5 (Bottom)	103.92	4012.99	0.03

8	D5 (Bottom)	42.43	882.32	0.05
9	DC1 (Bottom)	2.81	4274.86	0.00
10	D5 (Bottom)	87.63	4012.99	0.02
11	D5 (Max)	41.77	882.32	0.05
12	D5 (Max)	84.50	4012.99	0.02
13	D5 (Bottom)	20.93	882.32	0.02
14	D5 (Bottom)	29.72	4278.15	0.01
15	D5 (Max)	56.94	4012.99	0.01
16	DC1 (Bottom)	0.24	882.32	0.00
17	D1 (Bottom)	1.16	4278.15	0.00
18	D1 (Bottom)	1.10	4012.99	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	259.20	0.00
2	DC1 (Top)	0.00	907.20	0.00
3	DC1 (Top)	0.00	259.20	0.00
4	D1 (Max)	0.18	435.24	0.00
5	DC1 (Top)	0.00	907.20	0.00
6	DC1 (Top)	0.00	259.20	0.00
7	DC1 (Top)	0.00	907.20	0.00
8	DC1 (Top)	0.00	259.20	0.00
9	D1 (Max)	0.69	435.24	0.00
10	DC1 (Top)	0.00	907.20	0.00
11	DC1 (Top)	0.00	259.20	0.00
12	DC1 (Top)	0.00	907.20	0.00
13	DC1 (Top)	0.00	259.20	0.00
14	D1 (Max)	4.50	368.28	0.01
15	DC1 (Top)	0.00	907.20	0.00
16	DC1 (Top)	0.00	259.20	0.00
17	DC1 (Top)	0.00	368.28	0.00
18	DC1 (Top)	0.00	907.20	0.00

Shear

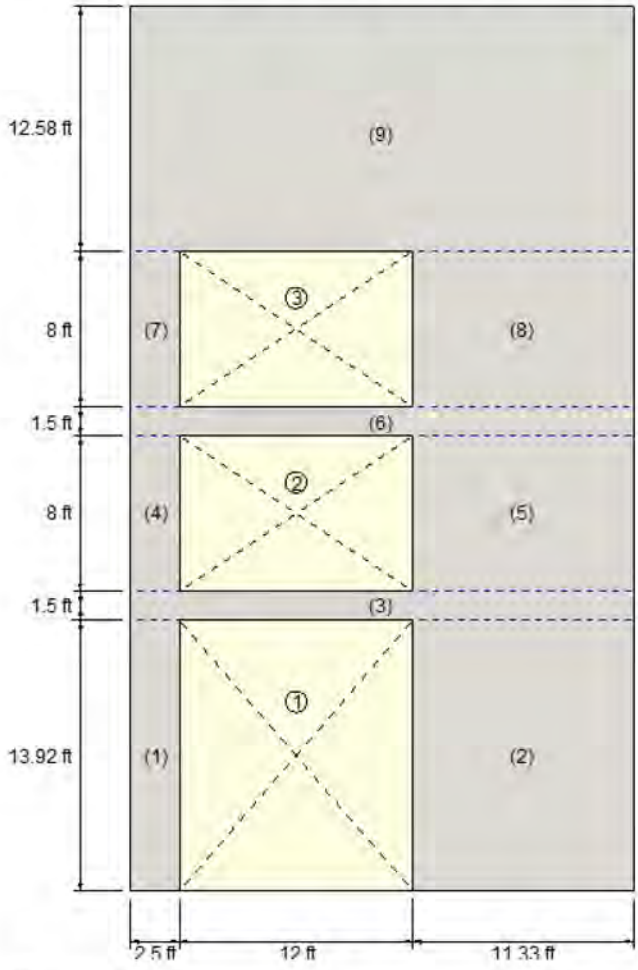
Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Bottom)	4.943	38.652	0.13
2	D6 (Bottom)	16.953	153.536	0.11
3	D6 (Bottom)	2.117	37.330	0.06
4	D4 (Top)	0.543	115.345	0.00
5	D5 (Bottom)	7.703	151.813	0.05
6	D5 (Bottom)	1.908	39.514	0.05
7	D5 (Bottom)	6.173	154.239	0.04
8	D5 (Bottom)	0.478	38.253	0.01
9	D4 (Top)	0.543	115.291	0.00
10	D6 (Top)	0.904	146.853	0.01
11	D6 (Top)	1.303	37.272	0.03
12	D6 (Top)	6.739	148.234	0.05
13	D5 (Top)	5.998	35.785	0.17
14	D5 (Bottom)	5.760	113.236	0.05
15	D6 (Top)	17.670	144.189	0.12
16	D6 (Bottom)	0.075	34.118	0.00
17	D6 (Bottom)	0.360	110.117	0.00
18	D6 (Max)	0.340	141.894	0.00

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.029	3.600	0.01
2	SC1 (Max)	0.012	3.600	0.00
3	SC1 (Max)	0.031	3.600	0.01
4	SC1 (Top)	0.000	3.600	0.00
5	SC1 (Max)	0.013	3.600	0.00
6	SC1 (Max)	0.046	3.600	0.01
7	SC1 (Max)	0.020	3.600	0.01
8	SC1 (Max)	0.049	3.600	0.01
9	SC1 (Top)	0.000	3.600	0.00
10	SC1 (Max)	0.021	3.600	0.01
11	SC1 (Max)	0.066	3.600	0.02
12	SC1 (Max)	0.027	3.600	0.01
13	SC1 (Max)	0.064	3.600	0.02
14	SC1 (Max)	0.024	3.600	0.01
15	SC1 (Max)	0.027	3.600	0.01
16	SC1 (Top)	0.000	0.040	0.00
17	SC1 (Top)	0.000	0.040	0.00
18	SC1 (Top)	0.000	0.040	0.00

Shear Wall Design

Status : Warnings in design
- Excessive spacing for shear reinforcement bars, Section 11.7.2.1 or 11.7.3.1 (Segment 9)
- Minimum ratio, horizontal reinforcement area vs. concrete, Table 11.6.1, Section 11.6.2 (Segment 9)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	2.50	13.92	Pier
2	14.50	0.00	11.33	13.92	Shear wall
3	0.00	13.92	25.83	1.50	Shear wall
4	0.00	15.42	2.50	8.00	Pier
5	14.50	15.42	11.33	8.00	Shear wall
6	0.00	23.42	25.83	1.50	Shear wall
7	0.00	24.92	2.50	8.00	Pier
8	14.50	24.92	11.33	8.00	Shear wall
9	0.00	32.92	25.83	12.08	Shear wall

Reinforcement

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	4-#7	9.00	33.89	22-#3	7.00	15.10
2	14-#7	10.00	33.89	22-#3	7.00	15.10
3	4-#7	9.00	33.89	2-#5	16.00	25.17
	13-#5	11.00	33.89	2-#5	16.00	25.17
	14-#7	10.00	33.89	2-#5	16.00	25.17
4	4-#7	9.00	33.89	14-#3	7.00	15.10
5	14-#7	10.00	33.89	14-#3	7.00	15.10
6	4-#7	9.00	33.89	2-#5	16.00	25.17
	13-#5	11.00	33.89	2-#5	16.00	25.17
	14-#7	10.00	33.89	2-#5	16.00	25.17
7	4-#7	9.00	33.89	14-#3	7.00	15.10
8	14-#7	10.00	33.89	14-#3	7.00	15.10
9	4-#7	9.00	33.89	8-#4	20.00	20.14
	11-#5	14.00	33.89	8-#4	20.00	20.14
	14-#7	10.00	33.89	8-#4	20.00	20.14

Intermediate results for axial-bending

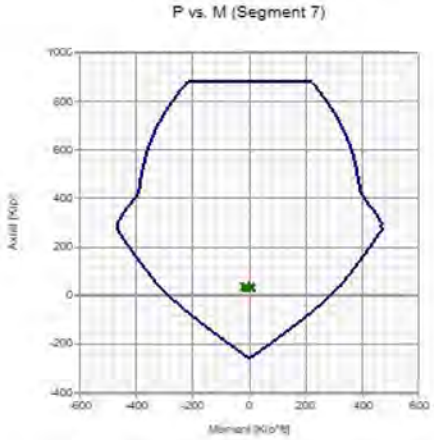
Segment	Condition	c [in]	d [in]
1	D5 (Bottom)	6.43	28.81
2	D5 (Top)	21.24	134.81
3	DC1 (Top)	36.84	308.81
4	D1 (Bottom)	6.20	28.81
5	D5 (Top)	20.83	134.81
6	D1 (Top)	36.15	308.81
7	D5 (Top)	6.14	28.81
8	D5 (Top)	20.43	134.81
9	DC1 (Top)	1.06	9.69

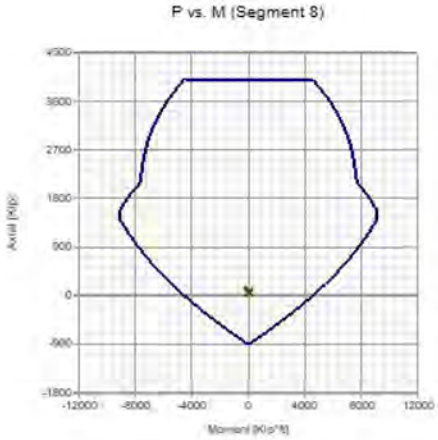
Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Bottom)	55.49	1.16	331.59	0.00
2	D5 (Top)	108.38	64.01	4989.18	0.01
3	DC1 (Top)	138.00	-166.75	21874.69	0.01
4	D1 (Bottom)	41.60	1.14	322.23	0.00
5	D5 (Top)	88.67	56.12	4907.31	0.01
6	D1 (Top)	108.62	-83.37	21566.73	0.00
7	D5 (Top)	38.40	-15.86	320.07	0.05
8	D5 (Top)	69.25	69.80	4826.65	0.01

9	DC1 (Top)	25.48	0.00	16369.55	0.00

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	55.49	882.32	0.06
2	D5 (Bottom)	134.93	4012.99	0.03
3	D5 (Bottom)	157.96	9170.18	0.02
4	D5 (Bottom)	47.53	882.32	0.05
5	D5 (Bottom)	103.92	4012.99	0.03
6	D5 (Bottom)	132.78	9170.18	0.01
7	D5 (Bottom)	41.77	882.32	0.05
8	D5 (Bottom)	84.50	4012.99	0.02
9	D5 (Bottom)	107.60	9173.47	0.01

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	259.20	0.00
2	DC1 (Top)	0.00	907.20	0.00
3	DC1 (Top)	0.00	1601.64	0.00
4	DC1 (Top)	0.00	259.20	0.00
5	DC1 (Top)	0.00	907.20	0.00
6	DC1 (Top)	0.00	1601.64	0.00
7	DC1 (Top)	0.00	259.20	0.00
8	DC1 (Top)	0.00	907.20	0.00
9	DC1 (Top)	0.00	1534.68	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D1 (Bottom)	0.119	81.643	0.00
2	DC1 (Top)	0.119	458.996	0.00
3	DC1 (Top)	0.000	1148.377	0.00
4	DC1 (Top)	0.209	81.643	0.00
5	D1 (Bottom)	0.209	458.996	0.00
6	DC1 (Top)	0.000	1148.377	0.00

7	D5 (Bottom)	2.694	81.643	0.03	
8	D5 (Bottom)	2.694	458.996	0.01	
9	DC1 (Top)	0.000	886.815	0.00	

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-1915.06	0.00	--
SC1	Right corner	1469.65	0.00	--

Notes

- Pu : Axial load
- Pn = Nominal axial load
- Mua = Moment at section
- Mu = Magnified moment at section
- Mcr = Cracking moment at section
- Mn = Maximum nominal moment
- Vu = Design shear force
- Vn = Nominal shear force

RAM Elements

Current Date: 8/14/2026 10:16 AM
Units system: English
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Design Results
Tilt-Up Wall

General Information

Global status : OK

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	44.00 [ft]
Total length	:	14.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	44.00	9.25

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.05	0.67
1	Wx	Vertical	0.05	0.67
1	DL	Vertical	0.05	0.67

Out-of-plane pressure loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	14.00	44.00

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	17-#6	10.00	28.46

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	45.343	0.029	0.240	0.12

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	7.94	0.87	1.02	7.63	0.90	379.53

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	11080.34	2846.86	3057.33

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-10.002

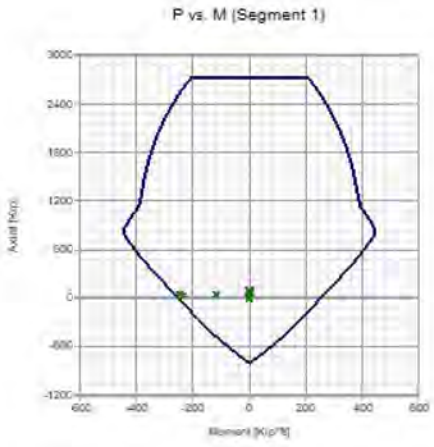
Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	45.34	-207.42	-246.72	264.54	0.93

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	DC1 (Top)	1.00	-0.65	94.70	252.58	0.37

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Bottom)	100.55	2721.02	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	807.84	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	18.742	110.469	0.17

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.004	3.520	0.00

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	14.00	44.00	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	17-#6	10.00	28.46	47-#4	11.00	24.67

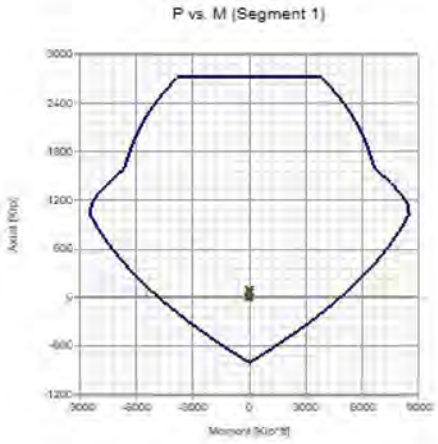
Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.23	7.63

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	1.00	0.00	4817.69	0.00

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Bottom)	100.55	2721.02	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	807.84	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	422.334	0.00

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-502.87	0.00	--
SC1	Right corner	502.87	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

Current Date: 8/14/2026 10:15 AM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T21_zone 5 corner.tup

Design Results
Tilt-Up Wall

General Information

Global status : Warnings in design

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	45.50 [ft]
Total length	:	25.83 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	45.00	11.25

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	2.50	0.00	20.00	13.92
Lower left	2.50	15.42	20.00	8.00
Lower left	2.50	24.92	20.00	8.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx

D5	Yes	1.2DL+LL+1.6Wx
D6	Yes	0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.55	0.67
1	Wx	Vertical	0.46	0.67
1	DL	Vertical	0.63	0.67

Out-of-plane pressure loads:

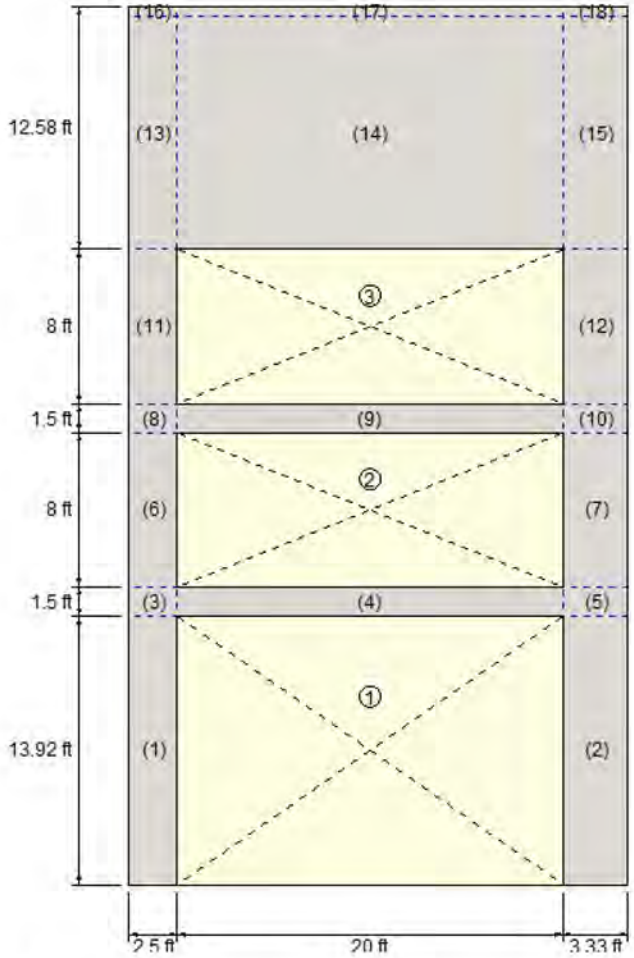
Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.05

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.05

Tilt-Up Walls Design

Status : Warnings in design
- The wall must be tension controlled, Section 11.8.1.1b (Segment 1)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.50	13.92
2	22.50	0.00	3.33	13.92
3	0.00	13.92	2.50	1.50
4	2.50	13.92	20.00	1.50
5	22.50	13.92	3.33	1.50
6	0.00	15.42	2.50	8.00
7	22.50	15.42	3.33	8.00
8	0.00	23.42	2.50	1.50
9	2.50	23.42	20.00	1.50
10	22.50	23.42	3.33	1.50
11	0.00	24.92	2.50	8.00
12	22.50	24.92	3.33	8.00
13	0.00	32.92	2.50	12.08
14	2.50	32.92	20.00	12.08
15	22.50	32.92	3.33	12.08
16	0.00	45.00	2.50	0.50
17	2.50	45.00	20.00	0.50
18	22.50	45.00	3.33	0.50

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#8	4.00	38.73
2	7-#8	6.00	38.73
3	7-#8	4.00	38.73
4	20-#5	12.00	19.36
5	7-#8	6.00	38.73
6	7-#8	4.00	38.73
7	7-#8	6.00	38.73
8	7-#8	4.00	38.73
9	20-#5	12.00	19.36
10	7-#8	6.00	38.73
11	7-#8	4.00	38.73
12	7-#8	6.00	38.73
13	7-#8	4.00	38.73
14	24-#5	10.00	19.36
15	7-#8	6.00	38.73
16	7-#8	4.00	38.73
17	24-#5	10.00	19.36
18	7-#8	6.00	38.73

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	65.931	0.195	0.360	0.54
2	D5 (Max)	70.504	0.157	0.360	0.44
3	D5 (Max)	63.457	0.188	0.360	0.52
4	D1 (Max)	-1.932	-0.001	0.360	0.00
5	D5 (Max)	67.809	0.151	0.360	0.42
6	D5 (Max)	59.403	0.176	0.360	0.49
7	D5 (Max)	62.635	0.139	0.360	0.39
8	D5 (Max)	57.342	0.170	0.360	0.47
9	D1 (Max)	-2.610	-0.001	0.360	0.00
10	D5 (Max)	60.386	0.134	0.360	0.37
11	D5 (Max)	55.254	0.164	0.360	0.45
12	D5 (Max)	59.678	0.133	0.360	0.37
13	D5 (Max)	27.951	0.083	0.360	0.23
14	D5 (Max)	21.452	0.008	0.360	0.02
15	D5 (Max)	31.067	0.069	0.360	0.19
16	D5 (Max)	0.207	0.001	0.360	0.00
17	D5 (Max)	1.657	0.001	0.360	0.00
18	D5 (Max)	0.276	0.001	0.360	0.00

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	6.17	2.83	3.78	9.63	0.87	288.42
2	D5 (Max)	6.22	2.15	2.86	9.63	0.90	331.67
3	D5 (Max)	6.16	2.81	3.75	9.38	0.86	270.41
4	D6 (Max)	6.21	0.30	0.41	9.56	0.90	4139.25
5	D5 (Top)	6.21	2.13	2.85	9.38	0.90	310.10
6	D5 (Top)	6.11	2.78	3.71	9.63	0.88	283.30
7	D5 (Top)	6.14	2.11	2.81	9.63	0.90	324.09
8	D5 (Top)	6.10	2.77	3.69	9.38	0.87	266.31
9	D6 (Max)	6.20	0.30	0.41	9.56	0.90	4139.25
10	D5 (Top)	6.13	2.09	2.79	9.38	0.90	304.45
11	D5 (Bottom)	6.07	2.76	3.68	9.63	0.89	282.34
12	D5 (Bottom)	6.11	2.09	2.79	9.63	0.90	323.36
13	D5 (Max)	5.81	2.56	3.41	9.50	0.90	268.74
14	D5 (Max)	7.65	0.38	0.51	9.69	0.90	4139.25
15	D5 (Max)	5.84	1.93	2.58	9.50	0.90	309.83
16	D6 (Max)	5.53	2.35	3.14	9.50	0.90	4190992.00
17	D6 (Max)	7.45	0.37	0.49	9.69	0.90	3.352793E07

18	D6 (Max)	5.53	1.73	2.31	9.50	0.90	5587933.00

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	3559.57	1925.32	1984.23
2	D5 (Max)	4746.05	2179.51	2281.74
3	D5 (Max)	3559.57	1808.42	1860.34
4	D6 (Max)	28476.56	3423.95	28476.56
5	D5 (Top)	4746.05	2045.67	2133.39
6	D5 (Top)	3559.57	1914.27	1949.04
7	D5 (Top)	4746.05	2169.41	2229.64
8	D5 (Top)	3559.57	1797.95	1832.10
9	D6 (Max)	28476.56	3421.20	28476.56
10	D5 (Top)	4746.05	2036.23	2094.54
11	D5 (Bottom)	3559.57	1908.45	1942.39
12	D5 (Bottom)	4746.05	2165.67	2224.63
13	D5 (Max)	3559.57	1811.07	1848.81
14	D5 (Max)	28476.56	4241.71	28476.56
15	D5 (Max)	4746.05	2065.12	2131.54
16	D6 (Max)	3559.57	1779.42	3559.57
17	D6 (Max)	28476.56	4152.00	28476.56
18	D6 (Max)	4746.05	2043.60	4746.05

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-5.645
2	D6 (Max)	-5.651
3	D6 (Max)	-6.475
4	D6 (Max)	-0.002
5	D6 (Max)	-6.491
6	D6 (Max)	-7.196
7	D6 (Max)	-7.226
8	D6 (Max)	-7.789
9	D6 (Max)	-0.002
10	D6 (Max)	-7.723
11	D6 (Max)	-7.178
12	D6 (Max)	-7.223
13	D6 (Max)	-6.335
14	D6 (Max)	-0.140
15	D6 (Max)	-6.340
16	D6 (Max)	0.000
17	D6 (Max)	0.000
18	D6 (Max)	0.000

Combined axial flexure

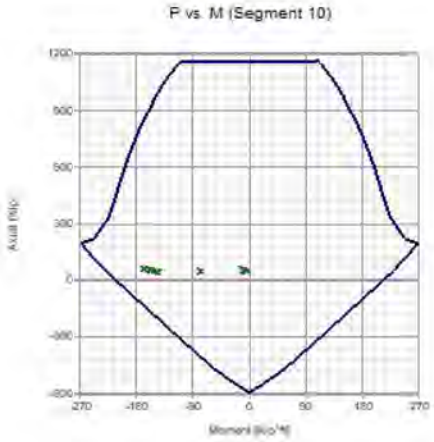
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	65.93	-80.32	-115.53	227.83	0.51
2	D5 (Max)	70.50	-94.61	-132.03	240.57	0.55
3	D5 (Max)	63.46	-85.75	-124.80	221.38	0.56
4	D6 (Max)	0.82	-0.42	-0.42	296.43	0.00
5	D5 (Top)	67.81	-101.30	-142.99	234.07	0.61
6	D5 (Top)	59.40	-103.98	-144.32	226.29	0.64
7	D5 (Top)	62.64	-122.08	-164.45	238.33	0.69
8	D5 (Top)	57.34	-105.23	-147.61	219.90	0.67
9	D6 (Max)	0.22	-0.42	-0.42	296.18	0.00
10	D5 (Top)	60.39	-123.33	-167.67	231.91	0.72

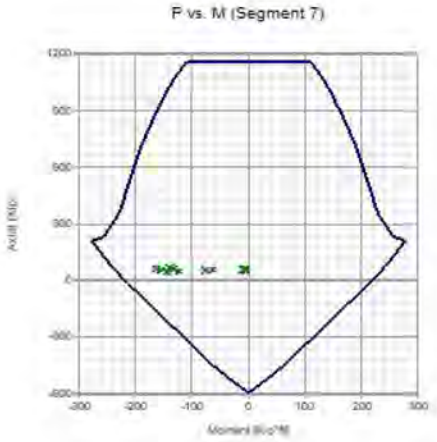
11	D5 (Bottom)	55.81	-105.24	-142.91	225.44	0.63
12	D5 (Bottom)	59.68	-123.34	-163.59	237.47	0.69
13	D5 (Max)	27.95	-100.20	-116.33	215.81	0.54
14	D5 (Max)	21.45	-42.41	-42.70	357.13	0.12
15	D5 (Max)	31.07	-115.94	-133.84	225.88	0.59
16	D6 (Max)	0.16	0.02	0.02	208.45	0.00
17	D6 (Max)	1.24	0.18	0.18	348.92	0.00
18	D6 (Max)	0.21	0.03	0.03	216.23	0.00

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D6 (Max)	46.55	-78.43	30.64	223.25	0.14
2	D6 (Max)	49.87	-92.56	40.85	234.49	0.17
3	D6 (Max)	44.73	-83.67	30.64	216.83	0.14
4	D1 (Max)	-1.93	0.00	245.09	295.28	0.83
5	D6 (Top)	47.88	-99.02	40.85	228.23	0.18
6	D6 (Max)	41.66	-100.81	30.64	222.09	0.14
7	D6 (Top)	43.97	-118.61	40.85	232.69	0.18
8	D6 (Max)	40.11	-101.87	30.64	215.71	0.14
9	DC1 (Top)	-2.61	0.00	245.09	294.99	0.83
10	D6 (Top)	42.29	-119.64	40.85	226.54	0.18
11	D6 (Top)	36.46	-95.81	30.64	220.79	0.14
12	D6 (Top)	38.37	-111.16	40.85	230.99	0.18
13	D6 (Top)	15.61	-2.23	30.64	212.54	0.14
14	DC1 (Top)	-8.26	-11.81	245.09	345.06	0.71
15	D6 (Top)	16.51	-2.96	40.85	221.41	0.18
16	D6 (Top)	0.00	0.00	30.64	208.41	0.15
17	D6 (Top)	0.02	0.00	245.09	348.42	0.70
18	D6 (Top)	0.00	0.00	40.85	216.16	0.19

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	71.79	865.72	0.08
2	D5 (Bottom)	78.31	1164.06	0.07
3	D5 (Bottom)	64.08	865.72	0.07
4	D1 (Bottom)	3.90	7127.52	0.00
5	D5 (Bottom)	68.64	1164.06	0.06
6	D5 (Bottom)	62.77	865.72	0.07
7	D5 (Bottom)	67.12	1164.06	0.06

8	D5 (Bottom)	57.97	865.72	0.07
9	DC1 (Bottom)	3.22	7127.52	0.00
10	D5 (Bottom)	61.22	1164.06	0.05
11	D5 (Bottom)	55.81	865.72	0.06
12	D5 (Max)	59.68	1164.06	0.05
13	D5 (Bottom)	27.95	865.72	0.03
14	D5 (Bottom)	48.58	7120.94	0.01
15	D5 (Max)	31.07	1164.06	0.03
16	DC1 (Bottom)	0.24	865.72	0.00
17	DC1 (Bottom)	1.93	7120.94	0.00
18	D1 (Bottom)	0.32	1164.06	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	597.24	0.00
2	DC1 (Top)	0.00	597.24	0.00
3	DC1 (Top)	0.00	597.24	0.00
4	D1 (Max)	1.93	669.60	0.00
5	DC1 (Top)	0.00	597.24	0.00
6	DC1 (Top)	0.00	597.24	0.00
7	DC1 (Top)	0.00	597.24	0.00
8	DC1 (Top)	0.00	597.24	0.00
9	D1 (Max)	2.61	669.60	0.00
10	DC1 (Top)	0.00	597.24	0.00
11	DC1 (Top)	0.00	597.24	0.00
12	DC1 (Top)	0.00	597.24	0.00
13	DC1 (Top)	0.00	597.24	0.00
14	D1 (Max)	8.26	803.52	0.01
15	DC1 (Top)	0.00	597.24	0.00
16	DC1 (Top)	0.00	597.24	0.00
17	DC1 (Top)	0.00	803.52	0.00
18	DC1 (Top)	0.00	597.24	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Bottom)	6.942	50.685	0.14
2	D6 (Bottom)	8.392	60.760	0.14
3	D6 (Bottom)	3.628	49.158	0.07
4	D4 (Top)	1.113	187.008	0.01
5	D6 (Bottom)	4.489	58.900	0.08
6	D5 (Bottom)	3.025	51.949	0.06
7	D5 (Bottom)	3.593	61.978	0.06
8	D5 (Bottom)	0.974	50.488	0.02
9	D4 (Top)	1.113	186.929	0.01
10	D5 (Bottom)	1.017	60.224	0.02
11	D6 (Top)	1.508	49.136	0.03
12	D6 (Top)	2.062	58.903	0.04
13	D5 (Top)	9.149	47.258	0.19
14	D5 (Bottom)	11.296	205.850	0.05
15	D5 (Top)	10.736	56.889	0.19
16	D6 (Bottom)	0.092	44.861	0.00
17	D6 (Max)	0.738	200.755	0.00
18	D6 (Bottom)	0.123	54.347	0.00

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax	
1	SC1 (Max)	0.041	3.600	0.01	
2	SC1 (Max)	0.033	3.600	0.01	
3	SC1 (Max)	0.045	3.600	0.01	
4	SC1 (Top)	0.000	3.600	0.00	
5	SC1 (Max)	0.036	3.600	0.01	
6	SC1 (Max)	0.067	3.600	0.02	
7	SC1 (Max)	0.055	3.600	0.02	
8	SC1 (Max)	0.071	3.600	0.02	
9	SC1 (Top)	0.000	3.600	0.00	
10	SC1 (Max)	0.058	3.600	0.02	
11	SC1 (Max)	0.094	3.600	0.03	
12	SC1 (Max)	0.076	3.600	0.02	
13	SC1 (Max)	0.091	3.600	0.03	
14	SC1 (Max)	0.024	3.600	0.01	
15	SC1 (Max)	0.074	3.600	0.02	
16	SC1 (Top)	0.000	0.040	0.00	
17	SC1 (Top)	0.000	0.040	0.00	
18	SC1 (Top)	0.000	0.040	0.00	

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

The following Tilt-Panel reports are failing. Refer to Tilt-Up retrofit calcs for HSS strongback fix for these panel types

Current Date: 7/29/2026 3:22 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T2_zone 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	42.50 [ft]
Total length	:	27.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	5.00	9.25
2	37.50	9.25

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
2	LL	Vertical	0.55	0.67
2	Wx	Vertical	0.46	0.67
2	DL	Vertical	0.63	0.67

Out-of-plane pressure loads:

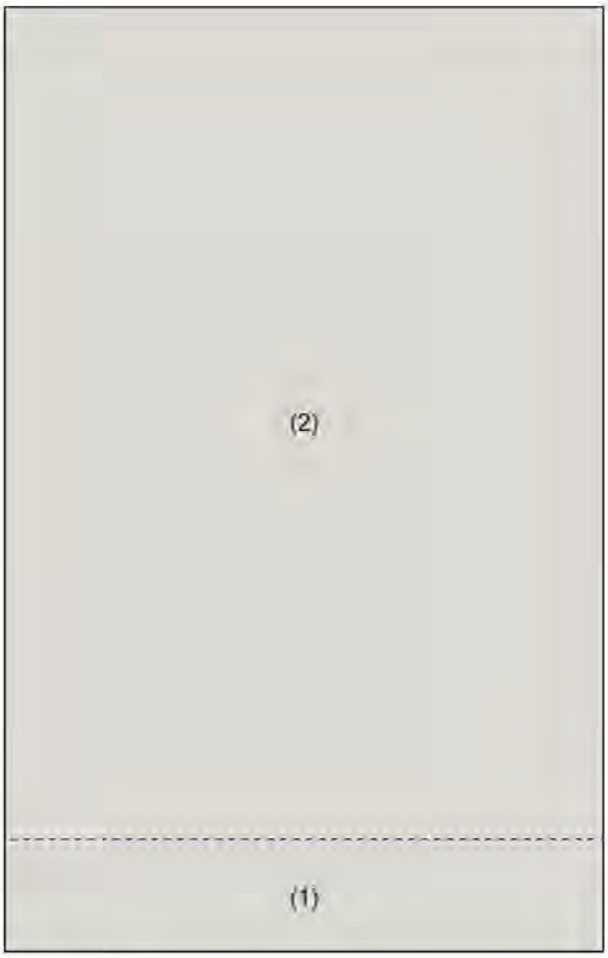
Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04
2	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 2)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	27.00	5.00
2	0.00	5.00	27.00	37.50

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	26-#5	12.00	23.72
2	26-#5	12.00	23.72

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	195.576	0.065	0.240	0.27
2	D5 (Max)	102.002	0.034	0.240	0.14

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D6 (Top)	9.47	0.57	0.67	7.69	0.90	44044.28
2	D5 (Bottom)	10.02	0.62	0.73	7.69	0.90	807.53

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D6 (Top)	21369.23	3786.79	4581.59
2	D5 (Bottom)	21369.23	3948.50	4725.04

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	0.097
2	D6 (Max)	-3.407

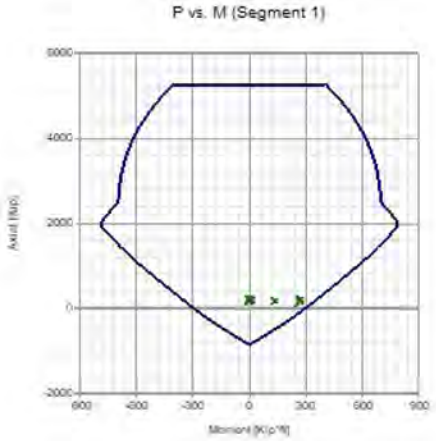
Combined axial flexure

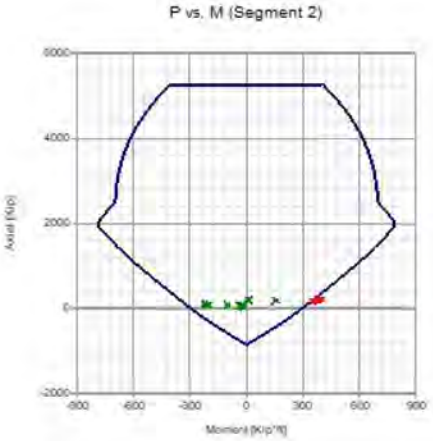
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D6 (Top)	140.51	265.17	266.30	339.74	0.78
2	D5 (Bottom)	195.51	271.18	400.45	355.78	1.13

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D6 (Max)	140.51	265.17	182.64	339.74	0.54
2	DC1 (Top)	23.96	-15.95	182.64	305.33	0.60

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	214.22	5270.20	0.04
2	D5 (Bottom)	195.51	5270.20	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	870.48	0.00
2	DC1 (Top)	0.00	870.48	0.00

Shear

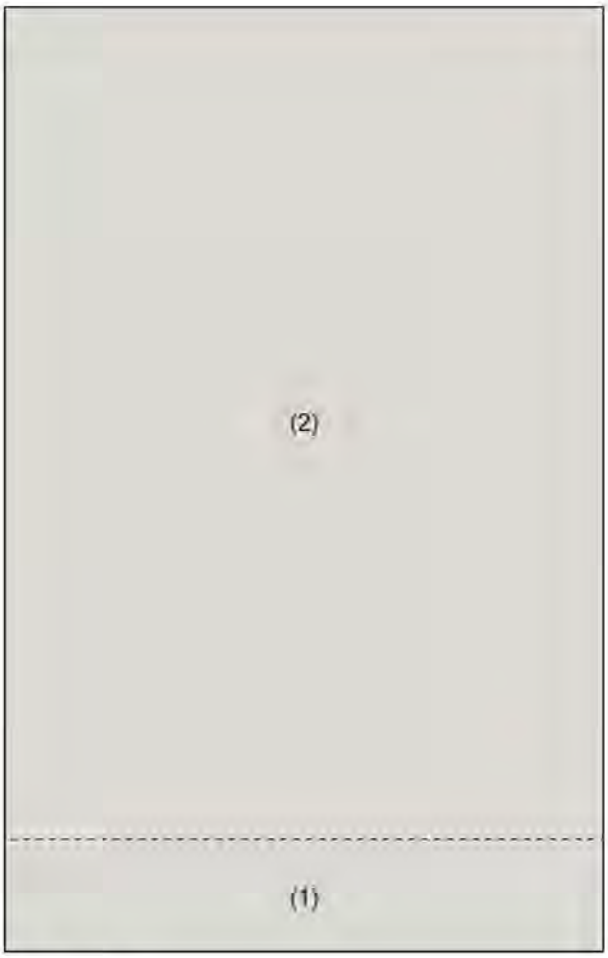
Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Max)	57.230	190.736	0.30
2	D6 (Bottom)	38.558	190.732	0.20

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.000	0.400	0.00
2	SC1 (Max)	0.038	3.000	0.01

Shear Wall Design

Status : Warnings in design
- Excessive spacing for shear reinforcement bars, Section 11.7.2.1 or 11.7.3.1 (Segment 1)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	27.00	5.00	Shear wall
2	0.00	5.00	27.00	37.50	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	26-#5	12.00	23.72	4-#4	19.00	24.67
2	26-#5	12.00	23.72	25-#4	18.00	24.67

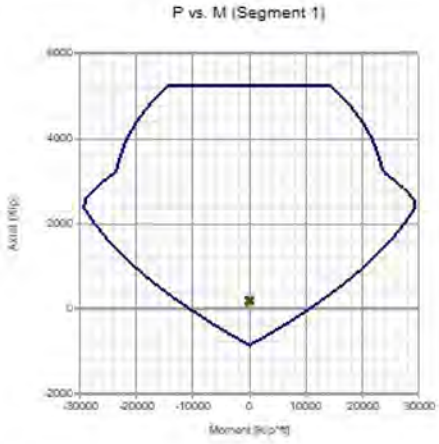
Intermediate results for axial-bending

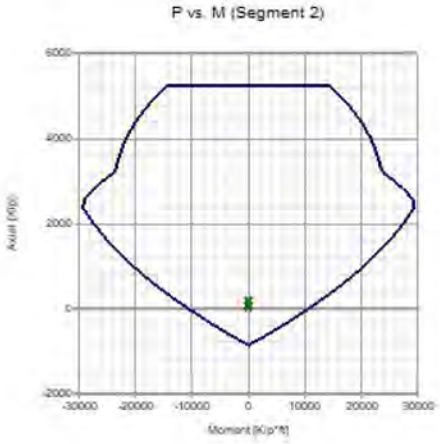
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	0.94	7.69
2	DC1 (Top)	0.80	7.69

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	187.66	0.00	12726.56	0.00
2	DC1 (Top)	23.96	0.00	10990.53	0.00

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	214.22	5270.20	0.04
2	D5 (Bottom)	195.51	5270.20	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	870.48	0.00
2	DC1 (Top)	0.00	870.48	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	733.428	0.00
2	DC1 (Top)	0.000	750.481	0.00

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-2019.73	0.00	--
SC1	Right corner	2019.73	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

Current Date: 7/29/2026 3:22 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T13_zone 5 corner.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	46.30 [ft]
Total length	:	20.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	39.50	9.25

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.55	0.67
1	Wx	Vertical	0.46	0.67
1	DL	Vertical	0.63	0.67

Out-of-plane pressure loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.05

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.05

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 1)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	20.00	39.50
2	0.00	39.50	20.00	6.80

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	20-#5	12.00	23.72
2	20-#5	12.00	23.72

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	114.499	0.052	0.240	0.21
2	D3 (Max)	18.824	0.008	0.240	0.04

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	7.35	0.60	0.70	7.69	0.90	487.37
2	D6 (Max)	6.34	0.47	0.56	7.69	0.90	82271.80

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	15829.06	2912.54	3164.01
2	D6 (Max)	15829.06	2607.79	15829.06

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-12.476
2	D6 (Max)	0.007

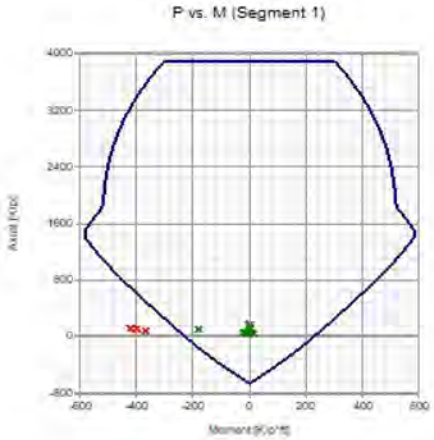
Combined axial flexure

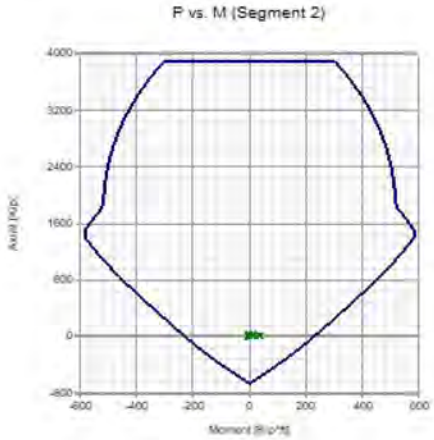
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	114.50	-288.99	-420.81	261.88	1.61
2	D6 (Max)	14.12	34.61	34.62	232.31	0.15

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	DC1 (Top)	39.74	-11.81	135.29	239.86	0.56
2	D6 (Top)	0.02	0.00	135.29	228.15	0.59

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	169.22	3903.04	0.04
2	DC1 (Bottom)	21.96	3903.04	0.01

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	669.60	0.00
2	DC1 (Top)	0.00	669.60	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	30.066	136.308	0.22
2	D6 (Bottom)	10.193	133.592	0.08

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.042	3.160	0.01
2	SC1 (Top)	0.000	0.544	0.00

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	20.00	39.50	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	20-#5	12.00	23.72	27-#4	18.00	24.67

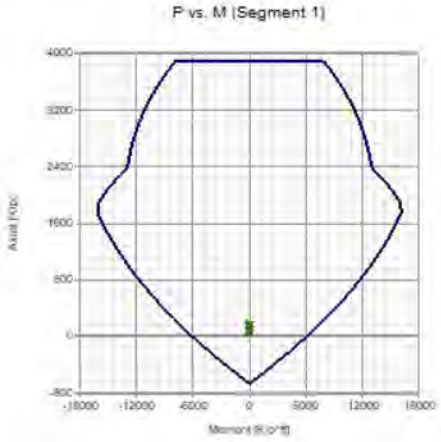
Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	0.84	7.69

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	39.74	0.00	6407.30	0.00

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	169.22	3903.04	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	669.60	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn	
1	DC1 (Top)	0.000	455.873	0.00	

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-1195.99	0.00	--
SC1	Right corner	1195.99	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

Current Date: 7/29/2026 3:19 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T23_zone 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	45.90 [ft]
Total length	:	27.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	45.90	9.25

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.55	0.67
1	Wx	Vertical	0.46	0.67
1	DL	Vertical	0.63	0.67

Out-of-plane pressure loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 1)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	27.00	45.90

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	33-#6	10.00	28.46

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	141.103	0.047	0.240	0.20

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	15.95	0.92	1.08	7.63	0.90	678.49

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	21369.23	5629.72	5947.80

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-13.280

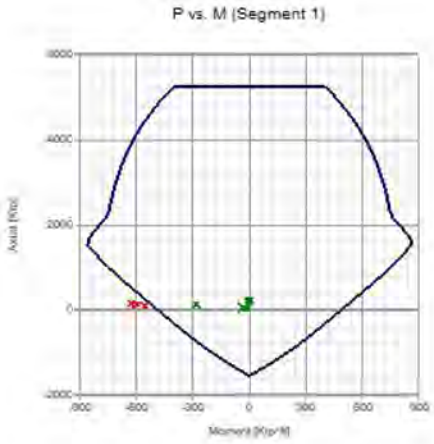
Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	141.10	-451.87	-625.24	527.29	1.19

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D1 (Max)	23.96	-15.95	182.64	495.75	0.37

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	226.94	5247.35	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	1568.16	0.00

Shear

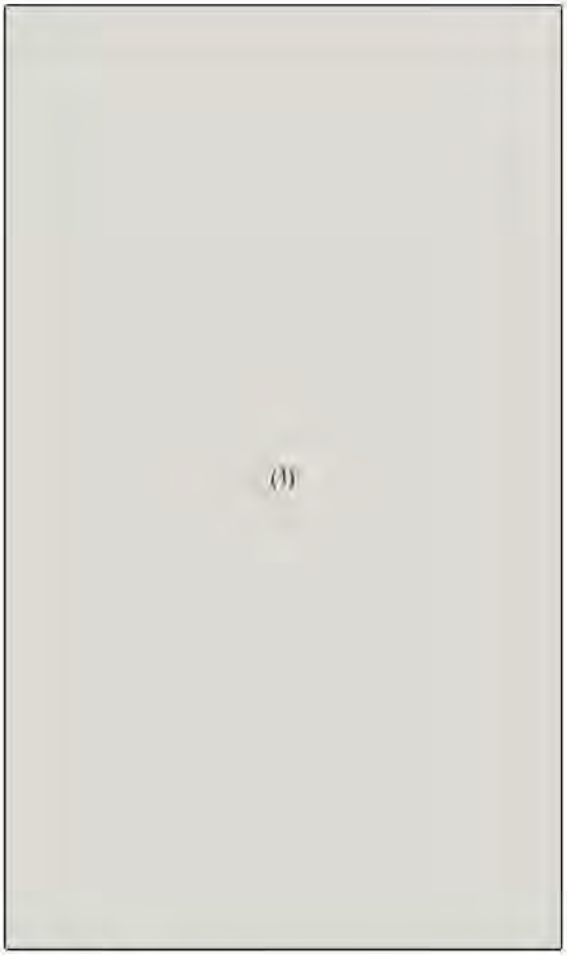
Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	37.248	216.794	0.17

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.056	3.672	0.02

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	27.00	45.90	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	33-#6	10.00	28.46	31-#4	18.00	24.67

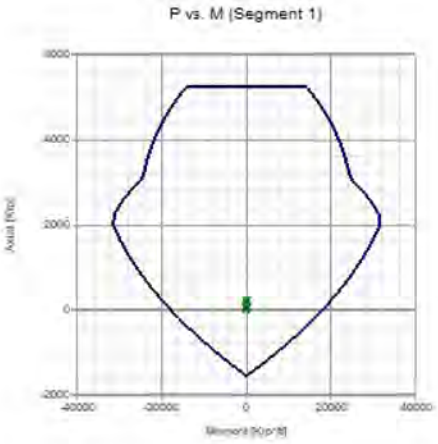
Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.26	7.63

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	23.96	0.00	18166.53	0.00

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	226.94	5247.35	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	1568.16	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	693.616	0.00

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-2162.85	0.00	--
SC1	Right corner	2162.85	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

Current Date: 7/29/2026 3:23 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T27_zone 5 corner.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	49.00 [ft]
Total length	:	35.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	5.00	9.25
2	38.00	9.25

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	15.83	5.00	3.33	7.20

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx

D6	Yes	0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
2	LL	Vertical	0.05	0.67
2	Wx	Vertical	0.05	0.67
2	DL	Vertical	0.05	0.67

Out-of-plane pressure loads:

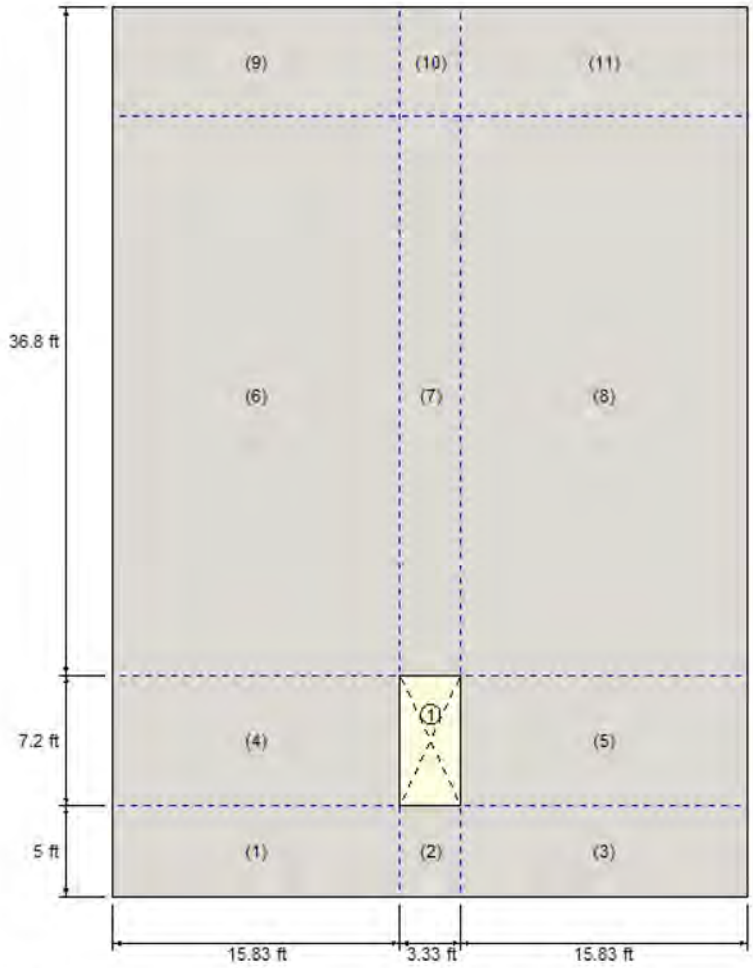
Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04
2	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 4)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	15.83	5.00
2	15.83	0.00	3.33	5.00
3	19.17	0.00	15.83	5.00
4	0.00	5.00	15.83	7.20
5	19.17	5.00	15.83	7.20
6	0.00	12.20	15.83	30.80
7	15.83	12.20	3.33	30.80
8	19.17	12.20	15.83	30.80
9	0.00	43.00	15.83	6.00
10	15.83	43.00	3.33	6.00
11	19.17	43.00	15.83	6.00

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	15-#5	13.00	19.36
2	4-#5	12.00	19.36
3	19-#5	10.00	19.36
4	15-#5	13.00	19.36
5	19-#5	10.00	19.36
6	15-#5	13.00	19.36
7	4-#5	12.00	19.36
8	19-#5	10.00	19.36

9	15-#5	13.00	19.36
10	4-#5	12.00	19.36
11	19-#5	10.00	19.36

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D1 (Max)	120.756	0.069	0.360	0.19
2	D5 (Max)	6.481	0.018	0.360	0.05
3	D1 (Max)	120.758	0.069	0.360	0.19
4	DC1 (Max)	123.755	0.070	0.360	0.20
5	D1 (Max)	123.757	0.070	0.360	0.20
6	D5 (Max)	52.106	0.030	0.360	0.08
7	D5 (Max)	6.234	0.017	0.360	0.05
8	D5 (Max)	52.107	0.030	0.360	0.08
9	D5 (Max)	13.147	0.007	0.360	0.02
10	D5 (Max)	2.767	0.007	0.360	0.02
11	D5 (Max)	13.147	0.007	0.360	0.02

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D6 (Top)	5.44	0.37	0.49	7.69	0.90	28114.27
2	D6 (Max)	1.29	0.39	0.52	7.69	0.90	31058.45
3	D6 (Top)	6.68	0.45	0.59	7.69	0.90	33287.24
4	D5 (Max)	5.74	0.40	0.53	7.69	0.90	506.01
5	D5 (Bottom)	6.98	0.48	0.64	7.69	0.90	593.58
6	D5 (Max)	5.17	0.34	0.46	7.69	0.90	757.79
7	D5 (Max)	1.30	0.40	0.53	7.69	0.90	163.27
8	D5 (Max)	6.41	0.42	0.56	7.69	0.90	881.81
9	D6 (Bottom)	4.75	0.30	0.40	7.69	0.90	102459.80
10	D6 (Max)	1.26	0.37	0.50	7.69	0.90	21568.37
11	D6 (Bottom)	5.99	0.37	0.50	7.69	0.90	102462.40

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D6 (Top)	12531.31	1857.96	2387.85
2	D6 (Max)	2637.91	436.90	2637.91
3	D6 (Top)	12531.63	2221.16	2827.21
4	D5 (Max)	12531.31	1938.22	2482.37
5	D5 (Bottom)	12531.63	2295.15	2911.96
6	D5 (Max)	12531.31	1782.84	3717.56
7	D5 (Max)	2637.91	440.59	800.99
8	D5 (Max)	12531.63	2151.90	4325.97
9	D6 (Bottom)	12531.31	1661.64	12531.31
10	D6 (Max)	2637.91	429.49	2637.91
11	D6 (Bottom)	12531.63	2040.12	12531.63

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	0.098
2	D6 (Max)	0.000
3	D6 (Max)	0.083
4	D6 (Max)	7.217
5	D6 (Max)	5.843
6	D6 (Max)	-2.570
7	D6 (Max)	-2.552
8	D6 (Max)	-2.176
9	D6 (Max)	0.003
10	D6 (Max)	0.003
11	D6 (Max)	0.003

Combined axial flexure

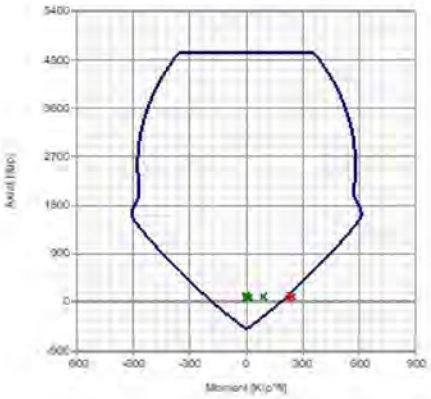
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D6 (Top)	78.99	171.32	171.97	205.94	0.84
2	D6 (Max)	4.85	-0.71	-0.71	48.24	0.01
3	D6 (Top)	79.00	171.32	171.87	246.00	0.70
4	D5 (Max)	108.35	171.73	240.35	214.89	1.12
5	D5 (Bottom)	108.35	171.73	226.97	254.84	0.89
6	D5 (Max)	52.11	-112.26	-123.60	197.75	0.63
7	D5 (Max)	6.23	-25.05	-26.39	48.66	0.54
8	D5 (Max)	52.11	-112.27	-121.87	237.91	0.51
9	D6 (Bottom)	9.86	19.33	19.33	184.82	0.10
10	D6 (Max)	2.08	4.07	4.07	47.41	0.09
11	D6 (Bottom)	9.86	19.33	19.33	225.19	0.09

Cracking moment

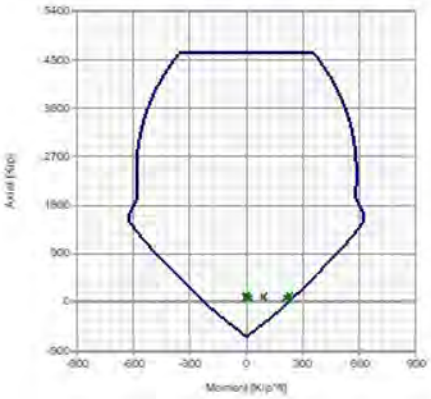
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D6 (Max)	78.99	171.32	131.17	205.94	0.64
2	D6 (Top)	3.98	0.00	27.61	47.98	0.58
3	D6 (Top)	79.00	171.32	131.18	246.00	0.53
4	D6 (Top)	69.14	13.48	131.17	202.94	0.65
5	D6 (Top)	69.14	13.48	131.18	243.04	0.54
6	D6 (Top)	13.47	17.92	131.17	185.97	0.71
7	DC1 (Top)	-1.40	-0.16	27.61	46.36	0.60
8	D6 (Top)	13.47	17.92	131.18	226.28	0.58
9	D6 (Top)	0.01	0.00	131.17	181.42	0.72
10	D6 (Top)	0.00	0.00	27.61	46.78	0.59
11	D6 (Top)	0.01	0.00	131.18	222.23	0.59

Interaction diagrams, P vs. M

P vs. M (Segment 4)



P vs. M (Segment 5)



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Bottom)	133.51	4636.22	0.03
2	DC1 (Bottom)	8.77	974.56	0.01
3	DC1 (Bottom)	133.52	4629.76	0.03
4	D1 (Bottom)	123.75	4636.22	0.03
5	D1 (Bottom)	123.76	4629.76	0.03
6	D1 (Bottom)	97.73	4636.22	0.02
7	D1 (Bottom)	15.18	974.56	0.02
8	D1 (Bottom)	97.73	4629.76	0.02
9	DC1 (Bottom)	15.34	4636.22	0.00
10	DC1 (Bottom)	3.23	974.56	0.00
11	DC1 (Bottom)	15.34	4629.76	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	502.20	0.00
2	DC1 (Top)	0.00	133.92	0.00
3	DC1 (Top)	0.00	636.12	0.00
4	DC1 (Top)	0.00	502.20	0.00
5	DC1 (Top)	0.00	636.12	0.00
6	DC1 (Top)	0.00	502.20	0.00
7	DC1 (Top)	1.40	133.92	0.01
8	DC1 (Top)	0.00	636.12	0.00
9	DC1 (Top)	0.00	502.20	0.00
10	DC1 (Top)	0.00	133.92	0.00
11	DC1 (Top)	0.00	636.12	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	37.009	134.025	0.28
2	D6 (Top)	0.565	29.072	0.02
3	D6 (Max)	37.009	144.343	0.26
4	D6 (Bottom)	25.853	134.229	0.19
5	D6 (Max)	25.854	144.547	0.18
6	D6 (Top)	16.725	127.218	0.13
7	D4 (Top)	3.611	28.554	0.13
8	D6 (Top)	16.726	137.535	0.12

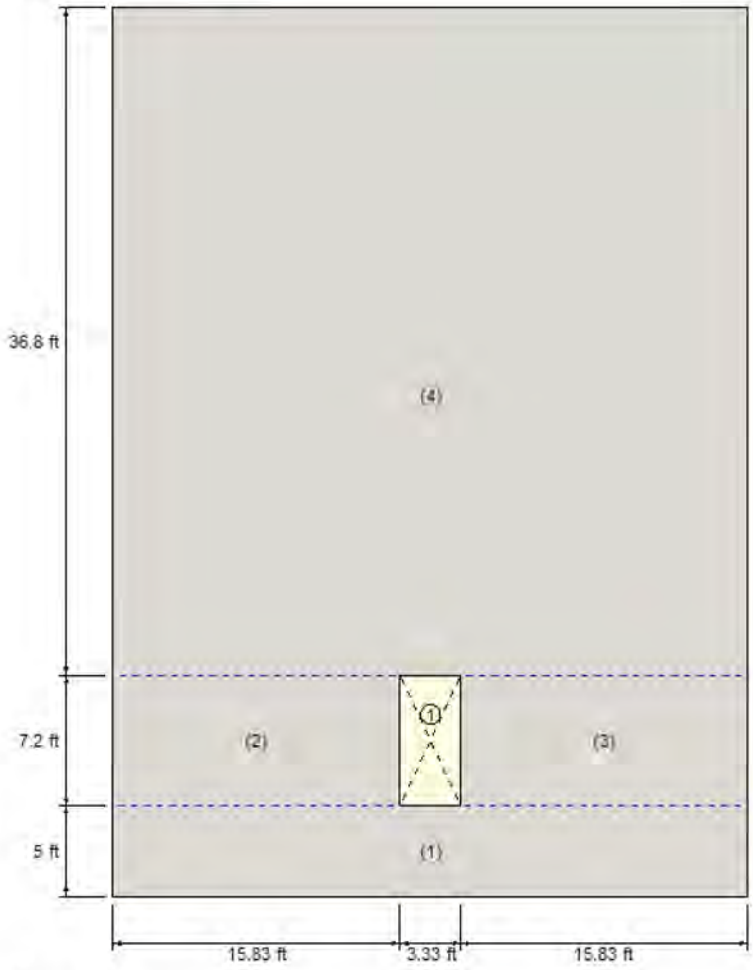
9	D6 (Max)	6.451	126.843	0.05	
10	D6 (Bottom)	1.358	28.874	0.05	
11	D6 (Max)	6.451	137.161	0.05	

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax	
1	SC1 (Max)	0.000	0.400	0.00	
2	SC1 (Top)	0.000	0.400	0.00	
3	SC1 (Max)	0.000	0.400	0.00	
4	SC1 (Max)	0.001	3.040	0.00	
5	SC1 (Max)	0.001	3.040	0.00	
6	SC1 (Max)	0.002	3.040	0.00	
7	SC1 (Max)	0.002	3.040	0.00	
8	SC1 (Max)	0.002	3.040	0.00	
9	SC1 (Top)	0.000	0.480	0.00	
10	SC1 (Top)	0.000	0.480	0.00	
11	SC1 (Top)	0.000	0.480	0.00	

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	35.00	5.00	Shear wall
2	0.00	5.00	15.83	7.20	Shear wall
3	19.17	5.00	15.83	7.20	Shear wall
4	0.00	12.20	35.00	30.80	Shear wall

Reinforcement

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	15-#5	13.00	19.36	4-#4	18.00	20.14
	4-#5	12.00	19.36	4-#4	18.00	20.14
	19-#5	10.00	19.36	4-#4	18.00	20.14
2	15-#5	13.00	19.36	5-#4	18.00	20.14
3	19-#5	10.00	19.36	5-#4	18.00	20.14
4	15-#5	13.00	19.36	21-#4	18.00	20.14
	4-#5	12.00	19.36	21-#4	18.00	20.14
	19-#5	10.00	19.36	21-#4	18.00	20.14

Intermediate results for axial-bending

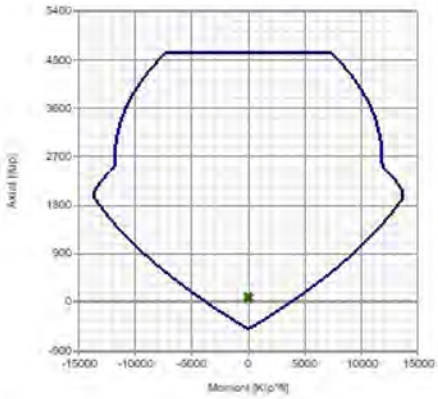
Segment	Condition	c [in]	d [in]
1	D1 (Top)	43.02	418.94
2	D1 (Top)	16.94	188.94
3	D1 (Top)	20.35	188.94
4	DC1 (Top)	0.69	7.69

Combined axial flexure

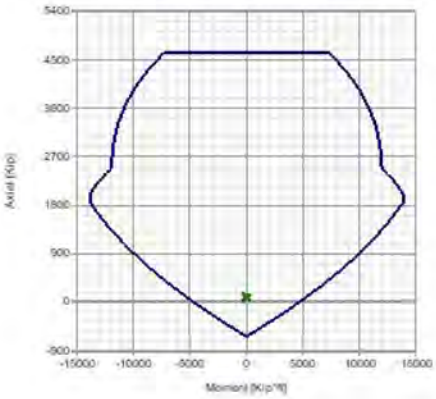
Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D1 (Top)	247.60	0.00	25662.70	0.00
2	D1 (Top)	105.37	-22.04	4459.57	0.00
3	D1 (Top)	105.37	22.04	5352.52	0.00
4	DC1 (Top)	36.45	0.00	19945.85	0.00

Interaction diagrams, P vs. M

P vs. M (Segment 2)



P vs. M (Segment 3)



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn	
1	DC1 (Bottom)	275.80	10240.54	0.03	
2	D1 (Bottom)	123.75	4636.22	0.03	
3	D1 (Bottom)	123.76	4629.76	0.03	
4	D1 (Bottom)	210.64	10240.54	0.02	

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn	
1	DC1 (Top)	0.00	1272.24	0.00	
2	DC1 (Top)	0.00	502.20	0.00	
3	DC1 (Top)	0.00	636.12	0.00	
4	DC1 (Top)	0.00	1272.24	0.00	

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn	
1	DC1 (Top)	0.000	1097.094	0.00	
2	DC1 (Top)	0.895	496.304	0.00	
3	D1 (Bottom)	0.895	496.316	0.00	
4	DC1 (Top)	0.000	1097.094	0.00	

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-3448.10	0.00	--
SC1	Right corner	3448.10	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

RAM Elements

Current Date: 7/29/2026 3:23 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T28a & T28b_zone 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	49.50 [ft]
Total length	:	7.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	2.00	12.00
2	43.50	12.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
2	DL	Vertical	0.05	0.00
2	LL	Vertical	0.05	0.00
2	Wx	Vertical	0.05	0.00

Out-of-plane pressure loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04
2	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : N. G.
- Excessive spacing for vertical bars, Section 11.7.2.1 or 11.7.3.1 (Segment 1)
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 2)
- Insufficient reinforcement to resist cracking moment, Section 11.8.1.1c (Segment 3)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	7.00	2.00
2	0.00	2.00	7.00	43.50
3	0.00	45.50	7.00	4.00

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	5-#5	20.00	23.72
2	5-#5	20.00	23.72
3	5-#5	20.00	23.72

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D1 (Max)	70.242	0.070	0.240	0.29
2	D5 (Max)	24.612	0.024	0.240	0.10
3	D3 (Max)	5.023	0.005	0.240	0.02

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D6 (Top)	1.99	0.49	0.57	10.44	0.90	130331.20
2	D5 (Bottom)	2.14	0.54	0.63	10.44	0.90	290.68
3	D6 (Max)	1.59	0.34	0.40	10.44	0.90	181691.80

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D6 (Top)	12096.00	1561.90	2169.18
2	D5 (Bottom)	12096.00	1657.82	2288.65
3	D6 (Max)	12096.00	1287.63	12096.00

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	0.012
2	D6 (Max)	-1.077
3	D6 (Max)	0.000

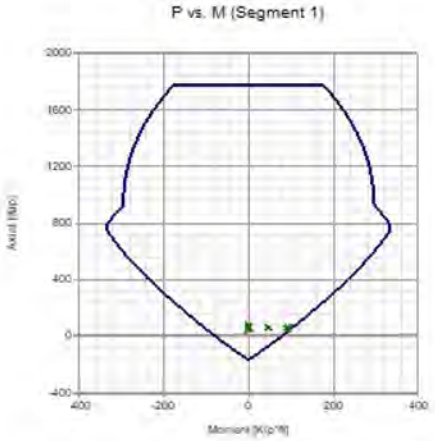
Combined axial flexure

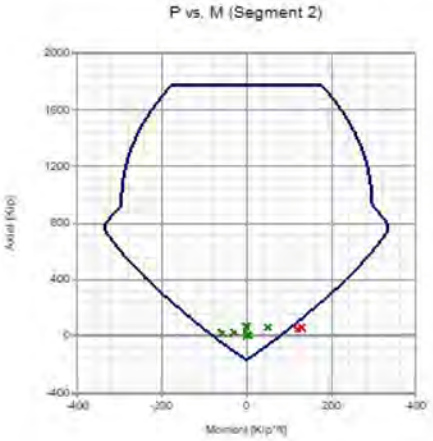
Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D6 (Top)	45.72	93.70	93.74	97.92	0.96
2	D5 (Bottom)	61.10	94.00	130.60	104.43	1.25
3	D6 (Max)	3.77	3.40	3.40	80.15	0.04

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	D6 (Top)	45.72	93.70	79.69	97.92	0.81
2	D6 (Top)	4.66	3.35	79.69	80.53	0.99
3	D6 (Top)	0.01	0.00	79.69	78.56	1.01

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Bottom)	73.15	1776.66	0.04
2	DC1 (Bottom)	70.22	1776.66	0.04
3	DC1 (Bottom)	5.86	1776.66	0.00

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	167.40	0.00
2	DC1 (Top)	0.00	167.40	0.00
3	DC1 (Top)	0.00	167.40	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Max)	47.471	55.111	0.86
2	D6 (Bottom)	11.362	55.109	0.21
3	D6 (Max)	1.703	50.550	0.03

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Top)	0.000	0.160	0.00
2	SC1 (Top)	0.000	3.480	0.00
3	SC1 (Top)	0.000	0.320	0.00

Shear Wall Design

Status : Warnings in design
- Excessive spacing for vertical bars, Section 11.7.2.1 or 11.7.3.1 (Segment 1)
- Excessive spacing for shear reinforcement bars, Section 11.7.2.1 or 11.7.3.1 (Segment 1)
- Minimum ratio, horizontal reinforcement area vs. concrete, Table 11.6.1, Section 11.6.2 (Segment 2)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	7.00	2.00	Shear wall
2	0.00	2.00	7.00	43.50	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	5-#5	20.00	23.72	2-#4	22.00	24.67
2	5-#5	20.00	23.72	29-#4	18.00	24.67

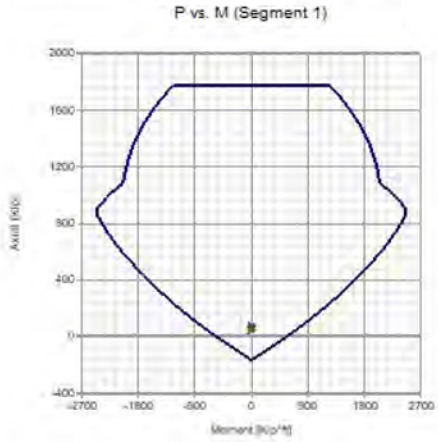
Intermediate results for axial-bending

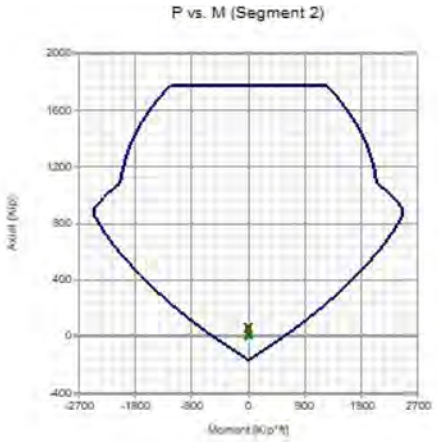
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	0.86	10.44
2	DC1 (Top)	0.60	10.44

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	70.24	0.00	778.03	0.00
2	DC1 (Top)	6.38	0.00	581.12	0.00

Interaction diagrams, P vs. M





Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D1 (Bottom)	73.15	1776.66	0.04
2	DC1 (Bottom)	70.22	1776.66	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	167.40	0.00
2	DC1 (Top)	0.00	167.40	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	212.168	0.00
2	DC1 (Top)	0.000	179.627	0.00

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-182.92	0.00	--
SC1	Right corner	182.92	0.00	--

Notes

- Pu : Axial load
- Pn = Nominal axial load
- Mua = Moment at section
- Mu = Magnified moment at section
- Mcr = Cracking moment at section
- Mn = Maximum nominal moment
- Vu = Design shear force
- Vn = Nominal shear force

RAM Elements

Current Date: 7/29/2026 3:24 PM
Units system: English
File name: Z:\Shared\Axiom Production\2026\A26-133 - Project Sunshine\Engineering\06 - Exterior Wall Design\Tilt Wall T30_zone 4.tup

Design Results
Tilt-Up Wall

General Information

Global status : N. G.

Design code	:	ACI 318-19
Analysis method	:	Simplified

Materials:

Material	:	C 4-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	4 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	3605 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Geometry

Total height	:	44.00 [ft]
Total length	:	14.00 [ft]
Foundation type	:	Continuous footing
Wall bottom restraint	:	Pinned

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]
1	44.00	9.25

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
Wx	No	WIND	Wind in X
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+1.6LL
D3	Yes		1.2DL+0.8Wx
D4	Yes		1.2DL+1.6Wx
D5	Yes		1.2DL+LL+1.6Wx
D6	Yes		0.9DL+1.6Wx

Loads

Consider Self Weight:

Load condition : DL

Distributed loads:

Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	LL	Vertical	0.55	0.67
1	Wx	Vertical	0.46	0.67
1	DL	Vertical	0.60	0.67

Out-of-plane pressure loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	0.04

Out-of-plane parapet pressure loads:

Condition	Magnitude [Kip/ft2]
Wx	0.04

Tilt-Up Walls Design

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 1)



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	14.00	44.00

Vertical reinforcement

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	17-#6	10.00	28.46

Vertical reinforcement

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	0.06*f´c [Kip/in2]	Ratio
1	D5 (Max)	70.767	0.046	0.240	0.19

Intermediate results for axial-bending

Segment	Condition	Ase [in2]	a [in]	c [in]	d [in]	ϕ	Kb [Kip]
1	D5 (Max)	8.20	0.91	1.07	7.63	0.90	384.43

Inertias

Segment	Condition	Ig [in4]	Icr [in4]	Ie [in4]
1	D5 (Max)	11080.34	2901.27	3096.75

Deflection

Segment	Condition	Δu [in]
1	D6 (Max)	-10.862

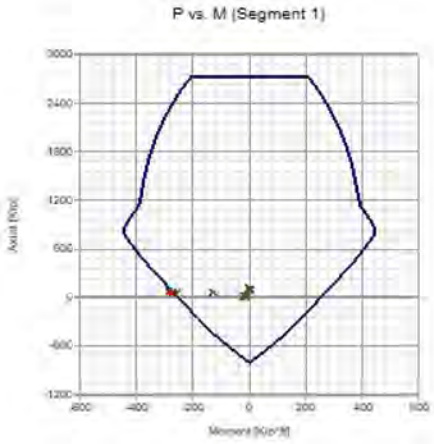
Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D5 (Max)	70.77	-215.89	-286.12	271.40	1.05

Cracking moment

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mcr [Kip*ft]	ϕ*Mn [Kip*ft]	Mcr/ϕ*Mn
1	DC1 (Top)	11.78	-7.84	94.70	255.49	0.37

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	113.43	2721.02	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	807.84	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	D6 (Top)	18.498	112.129	0.16

Deflection

Segment	Condition	Δs [in]	Δmax [in]	Δs/Δmax
1	SC1 (Max)	0.049	3.520	0.01

Shear Wall Design

Status : OK



Geometry

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]	Classification
1	0.00	0.00	14.00	44.00	Shear wall

Reinforcement

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	17-#6	10.00	28.46	47-#4	11.00	24.67

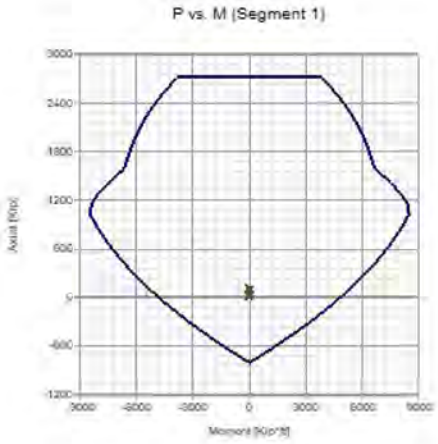
Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.25	7.63

Combined axial flexure

Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	11.78	0.00	4870.78	0.00

Interaction diagrams, P vs. M



Axial compression

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	D5 (Bottom)	113.43	2721.02	0.04

Axial tension

Segment	Condition	Pu [Kip]	ϕ*Pn [Kip]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	807.84	0.00

Shear

Segment	Condition	Vu [Kip]	ϕ*Vn [Kip]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	422.334	0.00

Stability Results

Status : OK

Global stability:

Safety factor : 1.5

Condition	Position	RM [Kip*ft]	OTM [Kip*ft]	FS
SC1	Left corner	-556.77	0.00	--
SC1	Right corner	556.77	0.00	--

Notes

* Pu : Axial load
* Pn = Nominal axial load
* Mua = Moment at section
* Mu = Magnified moment at section
* Mcr = Cracking moment at section
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force

TILT-UP PANEL RETROFIT:

-Utilize HSS strongbacks to reinforce tilt-up panels for out-of-plane wind flexure

-Tilt-up panels and HSS members to be compatible for deflection. HSS reinforcement to supplement moment capacity of walls

HSS12x8x3/8 @ 10' OC MAX
Es = 29,000 ksi

9 1/4" Tilt Panel
f'c = 4,000 psi
Ec = 3,605 ksi

WORST CASE WALL:
Tilt Panel #T13 - 39.75' TALL, ZONE 5 LOADING

-Check tilt panel (See RAM Elements Tilt-Up module results)
Flexural capacity at 1.61 DCR with full FM wind = 469 psf
Deflection = 12.48 in * 0.42 = 5.24 in

Panel works at 0.97 DCR with reduced wind = 291 psf, HSS can take excess load
Deflection (ultimate) = 6.5 in * 0.6 * 0.7 = 2.73"

-Check HSS12x8x3/8 strongback at 10' oc (See Enercalc results)
Deflection (service) = 3.45 in * 0.7 = 2.4in

Therefore deflections are compatible to share loading

ZONE 4 ENVELOPE CONDITION:
Tilt Panel #T23 - 43' TALL, ZONE 4 LOADING

-Check tilt panel (See RAM Elements Tilt-Up module results)
Flexural capacity at 1.19 DCR with full FM wind = 381 psf
Deflection = 13.28 in * 0.42 = 5.58 in

Panel works at 1.00 DCR with reduced wind =320 psf, HSS can take excess load
Deflection (ultimate) = 10.8 in * 0.6 * 0.7 = 4.5"

-Check HSS12x8x3/8 strongback at 10' oc (See Enercalc results)
Deflection (service) = 3.875 in * 0.7 = 2.72in

Therefore deflections are compatible to share loading

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: tilt up panel_HSS post reinforcement.ec6

LIC# : KW-06019020, Build:20.23.08.01 AXIOM (c) ENERCALC INC 1983-2023

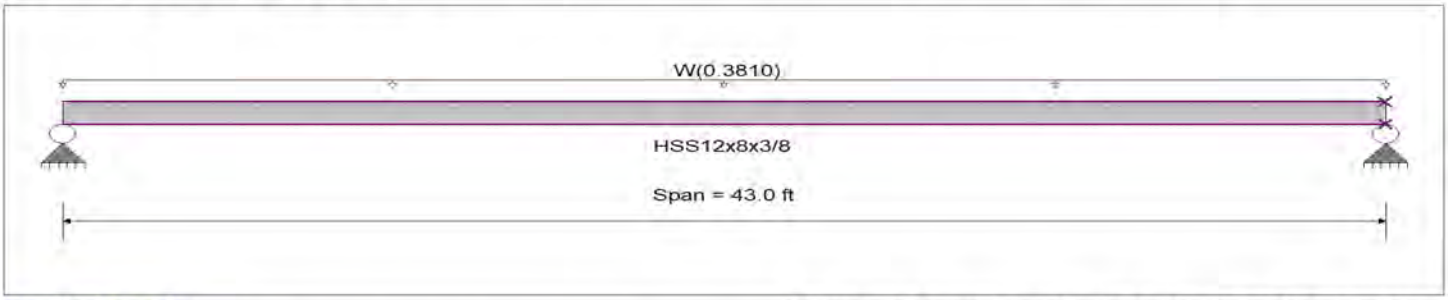
DESCRIPTION: ZONE 4 ENVELOPE CONDITION_HSS at 10' oc max

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.03810 ksf, Tributary Width = 10.0 ft, (FM GLOAB LWIND)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.709 : 1	Maximum Shear Stress Ratio =	0.063 : 1
Section used for this span	HSS12x8x3/8	Section used for this span	HSS12x8x3/8
Mu : Applied	140.894 k-ft	Vu : Applied	13.106 k
Mn * Phi : Allowable	198.750 k-ft	Vn * Phi : Allowable	206.420 k
Load Combination	+1.60W	Load Combination	+1.60W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	3.875 in	Ratio =	133	<180	Span: 1 : W Only
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 43.00 ft +0.80W	1		0.000				220.83	198.75	1.00	1.00	-0.00	229.36	206.42
Dsgn. L = 43.00 ft +1.60W	1	0.354	0.032	70.45		70.45	220.83	198.75	1.00	1.00	6.55	229.36	206.42
Dsgn. L = 43.00 ft	1	0.709	0.063	140.89		140.89	220.83	198.75	1.00	1.00	13.11	229.36	206.42

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	3.8749	21.623		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.192	8.192
Max Upward from Load Combinations	6.144	6.144
Max Upward from Load Cases	8.192	8.192
W Only	8.192	8.192
+0.750W	6.144	6.144

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: tilt up panel_HSS post reinforcement.ec6

LIC# : KW-06019020, Build:20.23.08.01 AXIOM (c) ENERCALC INC 1983-2023

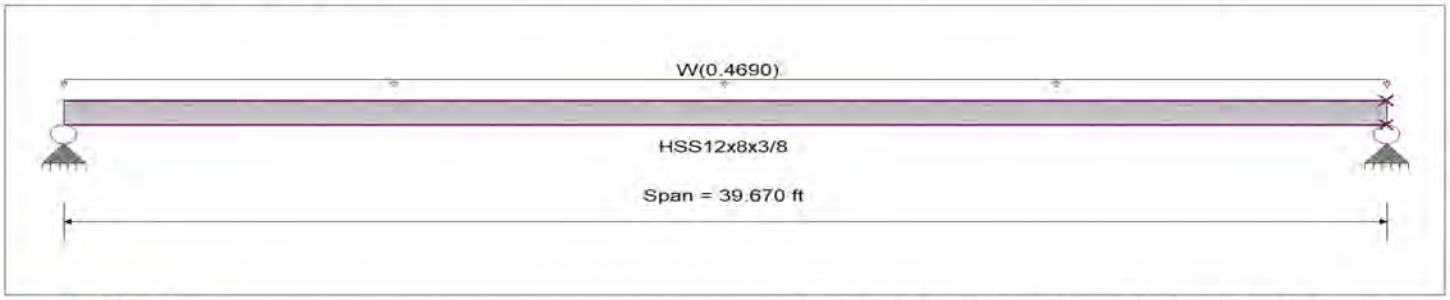
DESCRIPTION: ZONE 5 ENVELOPE CONDITION_HSS at 10' oc max

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.04690 ksf, Tributary Width = 10.0 ft, (FM GLOAB LWIND)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.743 : 1	Maximum Shear Stress Ratio =	0.072 : 1
Section used for this span	HSS12x8x3/8	Section used for this span	HSS12x8x3/8
Mu : Applied	147.614 k-ft	Vu : Applied	14.884 k
Mn * Phi : Allowable	198.750 k-ft	Vn * Phi : Allowable	206.420 k
Load Combination	+1.60W	Load Combination	+1.60W
		Location of maximum on span	39.670 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	3.455 in	Ratio =	138 <180	Span: 1 : W Only
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 39.67 ft +0.80W	1		0.000				220.83	198.75	1.00	1.00	-0.00	229.36	206.42
Dsgn. L = 39.67 ft +1.60W	1	0.371	0.036	73.81		73.81	220.83	198.75	1.00	1.00	7.44	229.36	206.42
Dsgn. L = 39.67 ft	1	0.743	0.072	147.61		147.61	220.83	198.75	1.00	1.00	14.88	229.36	206.42

Overall Maximum Deflections

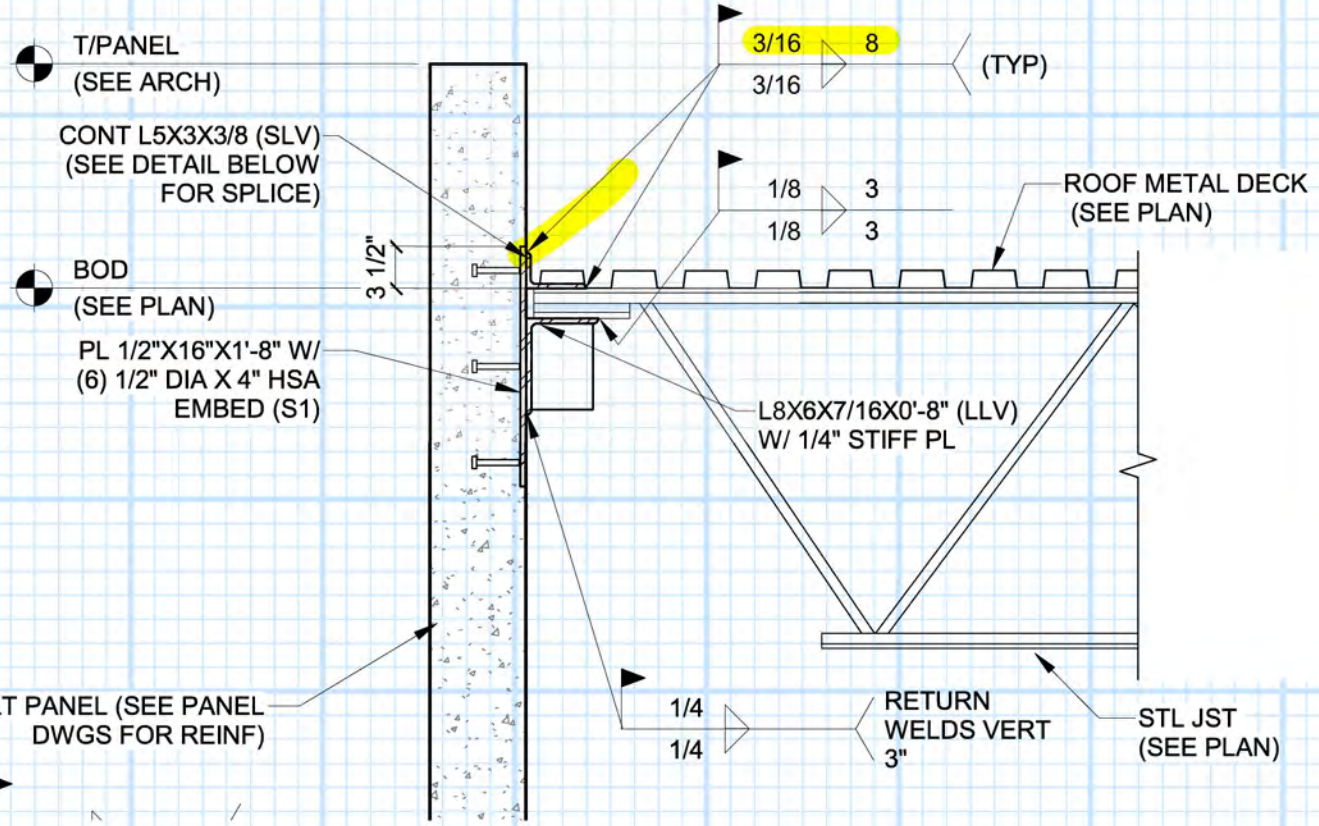
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	3.4553	19.948		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	9.303	9.303
Max Upward from Load Combinations	6.977	6.977
Max Upward from Load Cases	9.303	9.303
W Only	9.303	9.303
+0.750W	6.977	6.977

Summary of Tilt Panel diaphragm connection Loadings



SERVICE LEVEL WALL WIND PRESSURES

ZONE 4 = -60.9 PSF / 1.6 = 38.1 PSF

ZONE 5 = -74.95 PSF / 1.6 = 46.9 PSF

Use worst case (46.9 psf)

Diaphragm connections @ 6' OC

Walls =~ 45' tall

Reaction = 45' / 2 * 6' * 46.9 psf = 6.3kips
(service level wind)

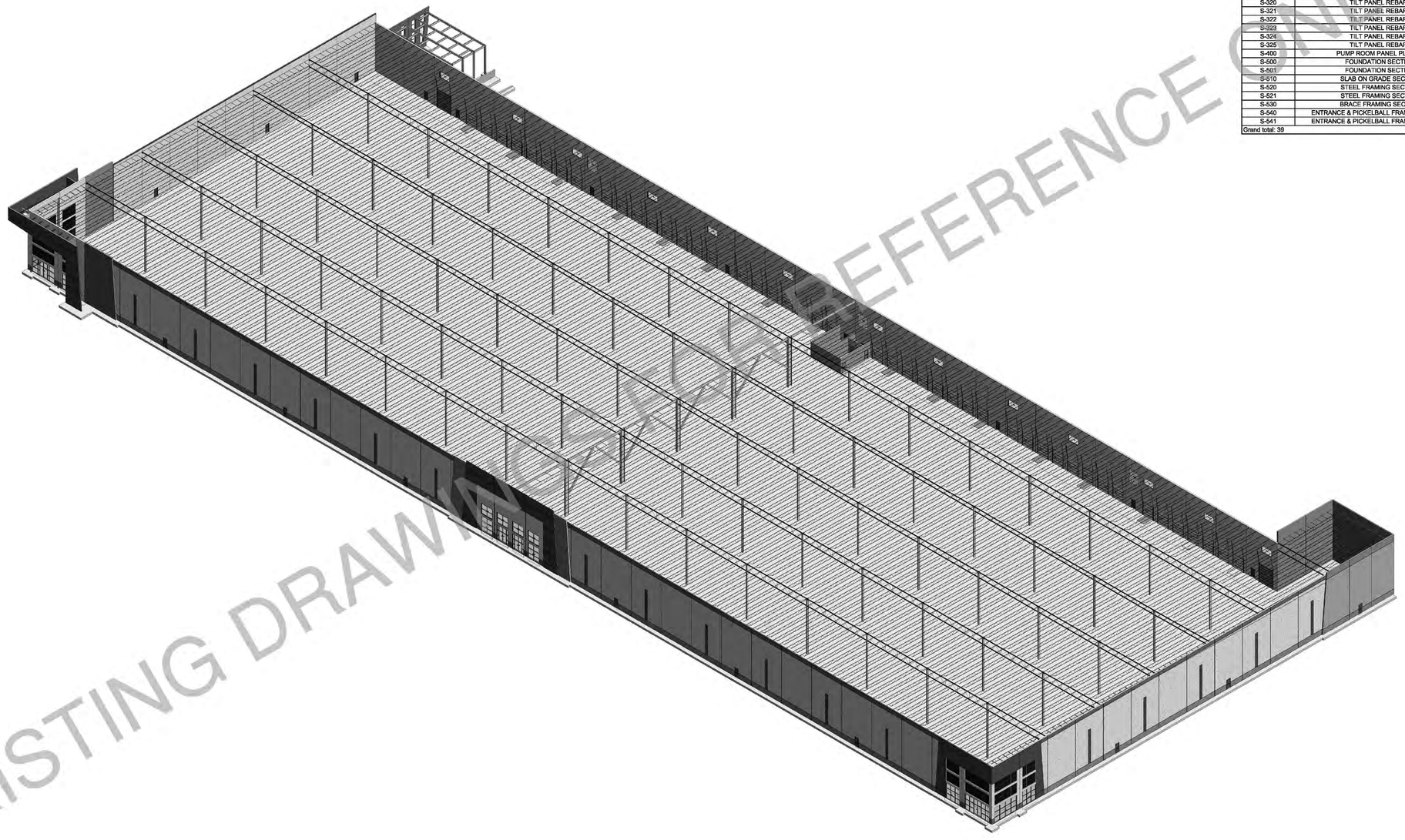
ASD fillet weld strength
Rn/Omega = 0.928 kip/in * D * L =
0.928 kip/in * 3 * 8in = 22.3 kip capacity

THEREFORE WELDS ARE OK

4.0 MISCELLANEOUS

5.0 APPENDIX

EXISTING DRAWING FOR REFERENCE ONLY



SHEET LIST	
SHEET NUMBER	SHEET NAME
S-000	COVER
S-100	GENERAL NOTES
S-101	GENERAL SCHEDULES
S-102	GRAVITY & LATERAL LOADING
S-103	BUILDING CLEAR HEIGHT ELEVATIONS
S-104	SPECIAL INSPECTIONS
S-201	OVERALL FOUNDATION PLAN
S-202	PARTIAL FOUNDATION PLAN - ZONE A
S-203	PARTIAL FOUNDATION PLAN - ZONE B
S-204	PARTIAL FOUNDATION PLAN - ENLARGED PLANS
S-211	OVERALL ROOF FRAMING PLAN
S-212	PARTIAL ROOF FRAMING PLAN - ZONE A
S-213	PARTIAL ROOF FRAMING PLAN - ZONE B
S-214	PARTIAL ROOF FRAMING PLAN - ENLARGED PLAN
S-300	OVERALL TILT PANEL PLAN
S-301	TILT PANEL EMBED/REINF SECTIONS & DETAILS
S-302	TILT PANEL EMBED/REINF SECTIONS & DETAILS
S-310	NORTH TILT PANEL ELEVATIONS
S-311	EAST TILT PANEL ELEVATIONS
S-312	EAST TILT PANEL ELEVATIONS
S-313	SOUTH TILT PANEL ELEVATIONS
S-314	WEST TILT PANEL ELEVATIONS
S-315	WEST TILT PANEL ELEVATIONS
S-316	WEST TILT PANEL ELEVATIONS
S-320	TILT PANEL REBAR ELEVATIONS
S-321	TILT PANEL REBAR ELEVATIONS
S-322	TILT PANEL REBAR ELEVATIONS
S-323	TILT PANEL REBAR ELEVATIONS
S-324	TILT PANEL REBAR ELEVATIONS
S-325	TILT PANEL REBAR ELEVATIONS
S-400	PUMP ROOM PANEL PLAN & ELEVATIONS
S-500	FOUNDATION SECTIONS & DETAILS
S-501	FOUNDATION SECTIONS & DETAILS
S-510	SLAB ON GRADE SECTIONS & DETAILS
S-520	STEEL FRAMING SECTIONS & DETAILS
S-521	STEEL FRAMING SECTIONS & DETAILS
S-530	BRACE FRAMING SECTIONS & DETAILS
S-540	ENTRANCE & PICKELBALL FRAMING SECTIONS & DETAILS
S-541	ENTRANCE & PICKELBALL FRAMING SECTIONS & DETAILS
Grand total: 39	

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PACE ROAD DISTRIBUTION CENTER - BUILDING 4

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PACE RD & PEARCE RD
AUBURNDALE, FL 33823

Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date	Job No.
2024 JUNE 28	24.0000849.000

Sheet Title

COVER

Sheet No.

S-000

CONSTRUCTION SET

9:105

PLOTTED: 2/26/2025 3:25:48 PM
SHEET NAME: COVER SHEET NUMBER: S-000 PROJECT NAME: PACE ROAD DISTRIBUTION CENTER - BUILDING 4
FILENAME: C:\Users\BlynchChamizo\Documents\24.03.058-Nelson-Pace Road DC-BLDG 4-R23_lychamizoRevitCU1.rvt

EXISTING DRAWINGS FOR REFERENCE ONLY

GENERAL:

1. DESIGN CODES:
   A. STRUCTURE DESIGNED IN ACCORDANCE WITH THE FLORIDA BUILDING CODE, 2023, 8TH EDITION.
   B. "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE" (ACI 318-19).
   C. AISC 360-16 SPECIFICATIONS FOR STRUCTURAL STEEL BUILDINGS.
   D. ASCE / SEI 7-22 MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES.
2. DESIGN LOADS:
   A. FOR GRAVITY AND LATERAL LOADS SEE SHEET S-102.
3. COORDINATE AND VERIFY FLOOR AND ROOF OPENING SIZES AND LOCATIONS WITH ARCHITECTURAL, MECHANICAL, PLUMBING AND ELECTRICAL DRAWINGS. FOR ADDITIONAL OPENINGS NOT SHOWN ON THE STRUCTURAL DRAWINGS SEE ARCHITECTURAL AND MECHANICAL DRAWINGS.
4. FOR DIMENSIONS NOT SHOWN SEE ARCHITECTURAL DRAWINGS.
5. SHOP DRAWING SUBMITTALS SHOULD BE TIMED BY THE CONTRACTOR AND VENDORS TO ALLOW FOR THE FOLLOWING:
   A. A MINIMUM OF 10 WORKING DAYS IN THE ENGINEER'S OFFICE. CONSULT WITH THE ARCHITECT FOR TIME REQUIRED IN THE ARCHITECT'S OFFICE BEFORE AND AFTER SUBMITTALS TO THE ENGINEER, SHIPPING TIME, ETC.
   B. TIME REQUIRED FOR REVISIONS TO THE SHOP DRAWINGS RESULTING FROM COMMENTS MADE ON THE SHOP DRAWINGS BY THE ENGINEER.
   C. TIME REQUIRED FOR RE-SUBMITTALS, IF NECESSARY. THIS OFFICE REQUESTS RE-SUBMITTALS ONLY IN RARE CASES.
6. REVIEW OF SUBMITTALS AND/OR SHOP DRAWINGS BY THE STRUCTURAL ENGINEER DOES NOT RELIEVE THE CONTRACTOR OF THE RESPONSIBILITY TO REVIEW AND CHECK SHOP DRAWINGS BEFORE SUBMITTAL TO THE STRUCTURAL ENGINEER. THE CONTRACTOR REMAINS SOLELY RESPONSIBLE FOR ERRORS AND OMISSION ASSOCIATED WITH THE PREPARATION OF SHOP DRAWINGS AS THEY PERTAIN TO MEMBER SIZES, DETAILS, AND DIMENSIONS SPECIFIED IN THE CONTRACT DOCUMENTS.
7. NOTIFY THE STRUCTURAL ENGINEER IN WRITING OF CONDITIONS ENCOUNTERED IN THE FIELD CONTRADICTORY TO THOSE SHOWN ON THE STRUCTURAL CONTRACT DOCUMENTS. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR THE DESIGN, ADEQUACY, AND SAFETY OF ERECTION BRACING, SHORING, TEMPORARY SUPPORTS, ETC.
8. CONTRACTOR SHALL FIELD VERIFY ALL EXISTING ABOVE AND BELOW GRADE CONDITIONS INCLUDING DIMENSIONS AND ELEVATIONS PRIOR TO FABRICATION AND CONSTRUCTION.
9. COORDINATE STRUCTURAL DRAWINGS WITH ARCHITECTURAL, MECHANICAL, ELECTRICAL, PLUMBING, AND CIVIL DRAWINGS. NOTIFY STRUCTURAL ENGINEER OF ANY CONFLICT AND/OR OMISSION.
10. GENERAL NOTES DO NOT REPLACE SPECIFICATIONS.
11. WHEN A SECTION IS CUT OR A DETAIL IS LABELED FOR A PARTICULAR CONDITION, THAT SECTION OR DETAIL SHALL APPLY FOR ALL SIMILAR CONDITIONS, REGARDLESS OF WHETHER CUT OR LABELED, UNLESS NOTED OTHERWISE.
12. CONTRACT DOCUMENTS SHALL GOVERN IN THE EVENT OF A CONFLICT WITH THE CODE OF PRACTICE OR SPECIFICATIONS OF ACI, PCI, AISC, SJI OR OTHER STANDARDS. WHERE A CONFLICT OCCURS WITHIN THE CONTRACT DOCUMENTS, THE STRICTEST REQUIREMENT SHALL GOVERN.
13. ELECTRONIC DRAWING FILES WILL NOT BE PROVIDED TO THE CONTRACTOR. REPRODUCTION OF STRUCTURAL DRAWINGS FOR SHOP DRAWINGS IS NOT PERMITTED.
14. STRUCTURAL ENGINEER IS NOT RESPONSIBLE FOR THE DESIGN OF STEEL STAIRS, HANDRAILS, CURTAIN WALL/WINDOW WALL SYSTEMS, COLD-FORMED METAL FRAMING, OR OTHER SYSTEMS NOT SHOWN IN THE STRUCTURAL DOCUMENTS. SUCH SYSTEMS SHALL BE DESIGNED, FURNISHED, AND INSTALLED AS REQUIRED BY OTHER CONSULTANTS IN THE CONTRACT DOCUMENTS.
15. IT IS EXPECTED THAT THE GENERAL CONTRACTOR IS EXPERIENCED IN THE TYPE OF CONSTRUCTION REQUIRED; THEREFORE IT IS EXPECTED THAT THE GENERAL CONTRACTOR WILL COORDINATE THE DRAWINGS WITH APPLICABLE ARCHITECTURAL, CIVIL, AND M/E/P DRAWINGS. THE CONSTRUCTION DOCUMENTS CONSIST OF THE ENTIRE SET OF DRAWINGS AND SPECIFICATIONS FROM ALL DISCIPLINES. THIS COORDINATION SHALL BE PERFORMED PRIOR TO ORDERING AND/OR FABRICATION OF ANY MATERIAL AND/OR THE CONSTRUCTION OF ANY ELEMENT.
16. PROVIDE CONTINGENCY FOR REPAIRING 2700 LINEAR FEET OF CRACKS IN THE FLOOR SYSTEM BY EPOXY GROUT INJECTION. UNUSED CONTINGENCY SHALL BE CREDITED TO THE OWNER.
17. SPECIAL INSPECTIONS SHALL BE IN ACCORDANCE WITH CHAPTER 17 OF THE BUILDING CODE AND SHALL BE MADE BY A QUALIFIED PERSON APPROVED BY THE BUILDING OFFICIAL. SPECIAL INSPECTOR SHALL OBSERVE WORK FOR CONFORMANCE WITH APPROVED DRAWINGS AND SPECIFICATIONS.
18. INSPECTION REPORTS SHALL BE FURNISHED TO THE BUILDING OFFICIAL, ARCHITECT AND STRUCTURAL ENGINEER. DISCREPANCIES SHALL BE BROUGHT TO THE ATTENTION OF THE CONTRACTOR, AND IF NOT CORRECTED, SHALL BE REPORTED TO THE BUILDING OFFICIAL, ARCHITECT AND STRUCTURAL ENGINEER.
19. SPECIAL INSPECTOR SHALL SUBMIT A FINAL REPORT STATING THAT THE STRUCTURAL WORK WAS, TO BEST OF HIS KNOWLEDGE, PERFORMED IN ACCORDANCE WITH THE APPROVED DRAWINGS, SPECIFICATIONS AND THE BUILDING CODE.
20. CONTRACTOR HAS SOLE RESPONSIBILITY FOR MEANS, METHODS, SAFETY, TECHNIQUES, SEQUENCES AND PROCEDURES OF CONSTRUCTION.
21. THE STRUCTURE IS STABLE ONLY IN ITS COMPLETED FORM. TEMPORARY SUPPORTS REQUIRED FOR STABILITY DURING ALL INTERMEDIATE STAGES OF CONSTRUCTION SHALL BE DESIGNED, FURNISHED AND INSTALLED BY THE CONTRACTOR. CONTRACTOR IS RESPONSIBLE FOR CONSTRUCTIBILITY ANALYSIS, AND ERECTION PROCEDURES, INCLUDING DESIGN AND ERECTION OF FALSEWORK, TEMPORARY BRACING, ETC.
22. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS WITH EDGE OF SLAB DIMENSIONS, OPENING LOCATIONS AND DIMENSIONS, DEPRESSED SLAB LOCATIONS AND EXTENTS, CURB LOCATIONS, AND CMU WALL LOCATIONS. ARCHITECT OR ENGINEER SHALL BE NOTIFIED OF ANY DISCREPANCY OR OMISSION.
23. STRUCTURAL DOCUMENTS ARE BEING RELEASED PRIOR TO DOCUMENTS BY OTHER DISCIPLINES (ARCHITECTURAL, MECHANICAL, ETC.). CONTRACTOR SHALL COORDINATE STRUCTURAL DOCUMENTS WITH OTHER PORTIONS OF THE CONTRACT DOCUMENTS AS THEY ARE RELEASED. ARCHITECT/STRUCTURAL ENGINEER SHALL BE NOTIFIED OF ANY DISCREPANCY OR OMISSION.

GEOTECHNICAL:

1. THE DESIGN SOIL BEARING PRESSURE IS 2500 PSF FOR COLUMN FOOTINGS AND WALL FOOTINGS BASED ON GEOTECHNICAL REPORT PREPARED BY UES DATED APRIL 30, 2024.
   A. EXTERIOR FOOTINGS AND THOSE NOT IN CLIMATE CONTROLLED AREAS SHOULD BEAR A MINIMUM OF 1 FOOT BELOW FINAL EXTERIOR GRADE.
   B. FLOOR SLAB IS DESIGNED WITH A MODULUS OF SUBGRADE REACTION OF 100 PCI, AND SHOULD BE FOUNDED ON A 8 INCH LAYER OF DENSE GRADED AGGREGATE.
2. GEOTECHNICAL ENGINEER SHALL VERIFY THE BEARING PRESSURE NOTED ABOVE, CONDITION AND/OR ADEQUACY OF ALL SUBGRADES, FILLS AND BACKFILLS BEFORE PLACEMENT OF FOUNDATIONS, FOOTINGS, SLABS, WALLS, FILLS, BACKFILLS, ETC.
3. THE CONTRACTOR SHALL FOLLOW ALL THE RECOMMENDATIONS OF THE SOIL REPORT.
4. SOIL WITHIN THE UPPER 2 FEET OF SUBGRADE BELOW INTERIOR CONCRETE SLABS ON GRADE AND ANY FILL WITHIN 5'-0" EACH SIDE OF BUILDING LIMIT SHALL BE COMPACTED TO 95% OF STANDARD PROCTOR. ALL FOOTINGS SHALL BEAR ON ORIGINAL UNDISTURBED SOIL OR CONTROLLED FILL.
5. SIDES OF FOUNDATIONS SHALL BE FORMED UNLESS CONDITIONS PERMIT EARTH FORMING. FOUNDATIONS POURED AGAINST THE EARTH REQUIRE THE FOLLOWING PRECAUTIONS:
   SLOPE SIDES OF EXCAVATIONS AS APPROVED BY GEOTECHNICAL ENGINEER AND CLEAN UP SLOUGHING BEFORE AND DURING CONCRETE PLACEMENT.
6. WHERE FOOTING STEPS ARE NECESSARY, THEY SHALL BE NO STEEPER THAN ONE VERTICAL TO TWO HORIZONTAL

DEFERRED SUBMITTALS:

1. DEFERRED SUBMITTALS INCLUDE THE FOLLOWING:
   A. STRUCTURAL STEEL CONNECTIONS.
   B. STRUCTURAL STEEL JOISTS AND JOIST GIRDERS.
   C. STAIR LANDINGS, GUARDRAILS, AND HANDRAILS.
   D. PREFABRICATED/PREMANUFACTURED CANOPIES AND AWNINGS, UNO, IN STRUCTURAL DRAWINGS.
   E. SUPPLEMENTARY PANEL LIFTING REINFORCING.
   F. COLD FORMED METAL FRAMING.
2. SHOP DRAWINGS AND CALCULATIONS FOR THE SUBMITTALS INDICATED ABOVE SHALL BE SUBMITTED FOR REVIEW AND APPROVAL. DEFERRED SUBMITTALS SHALL BE SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE PROJECT STATE.
3. REFER TO GENERAL NOTES, SECTIONS AND DETAILS FOR ADDITIONAL REQUIREMENTS FOR DEFERRED SUBMITTALS.
4. DO NOT BEGIN FABRICATION UNTIL DEFERRED SUBMITTALS HAVE BEEN REVIEWED.

REINFORCED CONCRETE:

1. CONCRETE SHALL BE NORMAL WEIGHT AND HAVE THE FOLLOWING MINIMUM 28 DAY STRENGTHS:

FOUNDATIONS:	3000 PSI TYPICAL (4000 PSI BRACED FRAMES)
SLAB-ON-GRADE:	4000 PSI (NO FLY ASH PERMITTED IN MIX DESIGN)
RETAINING WALLS:	3000 PSI (AIR ENTRAINED)
TILT-UP WALL PANELS (UNO):	4000 PSI UNO (SEE PANEL ELEVATIONS) (NO FLY ASH PERMITTED IN MIX DESIGN)

2. CONCRETE WALLS SHALL HAVE VERTICAL KEYED CONSTRUCTION OR TOOLED CONTROL JOINTS AT 20'-0" OC MAXIMUM. STOP HORIZONTAL REINFORCEMENT 2" CLEAR OF EACH SIDE OF CONSTRUCTION JOINTS. ALLOW ONLY 50% OF HORIZONTAL WALL REINFORCING TO PASS THROUGH CONTROL JOINTS. JOINTS ARE REQUIRED IN THE WALLS BUT NOT IN THE FOOTINGS SUPPORTING THE WALLS
3. HORIZONTAL REINFORCEMENT IN FOOTINGS, TURNDOWN SLABS AND WALLS SHALL BE CONTINUOUS AROUND CORNERS. WHERE HORIZONTAL REINFORCEMENT IS INDICATED ONLY ON INTERIOR OR EXTERIOR FACE OF WALLS, THESE BARS SHALL BE CONTINUOUS AROUND BENDS AND CORNERS. ADDITIONAL HORIZONTAL CORNER BARS OF SAME SIZE AND SPACING SHALL BE PROVIDED AT OPPOSITE FACE. UNLESS NOTED OTHERWISE ON THE DRAWINGS.
4. CONCRETE PROTECTION FOR REINFORCEMENT, SEE TABLE.
5. DETAILING, FABRICATION AND PLACING OF REINFORCING STEEL SHALL BE IN ACCORDANCE WITH ACI 315-99, "DETAILS AND DETAILING OF CONCRETE REINFORCEMENT - 1999".
6. THE PROPOSED MATERIALS AND MIX DESIGN SHALL BE FULLY DOCUMENTED AND REVIEWED BY THE OWNER'S TESTING LABORATORY. RESPONSIBILITY FOR OBTAINING THE REQUIRED DESIGN STRENGTH IS THE CONTRACTOR'S.
7. USE OF CALCIUM CHLORIDE, CHLORIDE IONS, OR OTHER SALTS IN CONCRETE IS NOT PERMITTED.
8. CHAMFER OR ROUND ALL EXPOSED CORNERS A MINIMUM OF 3/4".
9. TIE ALL REINFORCING STEEL AND EMBEDMENTS SECURELY IN PLACE PRIOR TO PLACING CONCRETE. PROVIDE SUFFICIENT SUPPORTS TO MAINTAIN THE POSITION OF REINFORCEMENT WITHIN SPECIFIED TOLERANCES DURING ALL CONSTRUCTION ACTIVITIES.
10. UNLESS NOTED OTHERWISE LAP, SPLICE, OR EMBEDMENT LENGTHS SHALL CONFORM TO ACI-318 CLASS B TENSION SPLICE, SEE TABLE.
11. NO BACKFILL SHALL BE PLACED AGAINST CONCRETE OR CMU WALLS UNTIL CONCRETE (AND/OR MORTAR, WHERE OCCURS) HAS ATTAINED FULL STRENGTH AND WALLS HAVE BEEN BRACED AGAINST LATERAL THRUST - SUCH BRACING SHALL REMAIN IN PLACE UNTIL SLAB ON EARTH (AND SUPPORTED SLAB, IF ANY) HAS BEEN PLACED AND ALLOWED TO CURE FOR 28 DAYS.
12. DO NOT WELD OR TACK WELD REINFORCING STEEL UNLESS APPROVED OR DIRECTED BY THE STRUCTURAL ENGINEER.
13. HORIZONTAL CONSTRUCTION JOINTS ARE PERMITTED ONLY WHERE INDICATED. THE LOCATION OF VERTICAL CONSTRUCTION JOINTS SHALL BE APPROVED BY THE STRUCTURAL ENGINEER. CONSTRUCTION JOINTS SHALL BE THOROUGHLY ROUGHENED BY MECHANICAL MEANS AND CLEANED.
14. DETAIL CONCRETE REINFORCEMENT AND ACCESSORIES IN ACCORDANCE WITH ACI 315 DETAILING MANUAL. SUBMIT SHOP DRAWINGS FOR APPROVAL, SHOWING ALL FABRICATION DIMENSIONS AND LOCATIONS OF REINFORCING STEEL AND ACCESSORIES. DO NOT BEGIN FABRICATION UNTIL SHOP DRAWINGS ARE COMPLETED AND REVIEWED.
15. REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60 UNLESS NOTED OTHERWISE.
16. WELDED WIRE FABRIC (MESH) SHALL CONFORM TO ASTM A1064 AND SHALL BE PROVIDED IN FLAT SHEETS (ROLLS ARE NOT PERMITTED).
17. PROVIDE CONTINUOUS REINFORCEMENT WHEREVER POSSIBLE; SPLICE ONLY AS SHOWN OR APPROVED; STAGGER SPLICES WHERE POSSIBLE; USE TENSION SPLICE (CLASS "B") UNLESS NOTED OTHERWISE.
18. SUBMIT STRUCTURAL DRAWINGS WHICH ADEQUATELY DEPICT THE REINFORCING BAR SIZES AND PLACEMENT. WRITTEN DESCRIPTION OF REINFORCEMENT WITHOUT ADEQUATE SECTIONS, ELEVATIONS, AND DETAILS IS NOT ACCEPTABLE.
19. PROVIDE DOWELS FROM FOUNDATIONS THE SAME SIZE AND NUMBER AS THE VERTICAL WALL OR COLUMN REINFORCING, UNLESS NOTED OTHERWISE.
20. RESULTS OF CONCRETE COMPRESSIVE STRENGTH TESTS SHALL BE AVAILABLE AT THE SITE FOR BUILDING OFFICIAL'S REVIEW.

MASONRY:

1. CONCRETE MASONRY DESIGN AND CONSTRUCTION SHALL CONFORM TO: ACI 530/ASCE 5, BUILDING CODE REQUIREMENTS FOR CONCRETE MASONRY STRUCTURES AND ACI SEQ MANUAL VOLUME 6, SPECIFICATION FOR CONCRETE MASONRY CONSTRUCTION.
2. PROVIDE TYPE S MORTAR IN ACCORDANCE WITH ASTM C270, UNLESS NOTED OTHERWISE. MORTAR BED JOINTS SHALL NOT EXCEED 5/8" THICKNESS.
3. LAY MASONRY UNITS IN RUNNING BOND UNLESS NOTED OTHERWISE.
4. PROVIDE HORIZONTAL JOINT REINFORCEMENT COMPLYING WITH ASTM A82, NO. 9 GAUGE OR HEAVIER, ZINC COATED, OR W1.7 HORIZONTAL JOINT LADDER REINFORCING PLACED 16" ON CENTER UNLESS NOTED OTHERWISE. PROVIDE SPECIAL CONFIGURATIONS FOR CORNERS AND INTERSECTIONS. PLACE REINFORCEMENT ABOVE ALL WALL OPENINGS AT FIRST COURSE ABOVE OPENING.
5. PROVIDE CONTROL JOINTS IN ALL CONCRETE MASONRY WALLS AT LOCATIONS APPROVED BY THE ARCHITECT AT A MAXIMUM SPACING OF 3 TIMES THE WALL HEIGHT OR 40'-0", WHICHEVER IS LESS.
6. PROVIDE MINIMUM (2)#4 VERTICAL WITH GROUT FULL HEIGHT AT EACH SIDE OF OPENINGS PROVIDE (1)#5 WITH GROUT FULL HEIGHT IN TWO CORNER CELLS OF END WALLS. PERIMETER MASONRY WALL SHALL BE REINFORCED WITH #5 @ 24" VERTICAL UNLESS NOTED OTHERWISE.
7. MASONRY REINFORCING STEEL SHALL BE PLACED IN THE CENTER OF THE WALL UNLESS NOTED OTHERWISE. LAP, SPLICE, OR EMBEDMENT LENGTHS SHALL CONFORM TO ACI 530 DEVELOPMENT LENGTHS.
8. ALL MASONRY WALLS TO BE FULLY GROUTED, BELOW GRADE, AS OCCURS.
9. WHERE CMU WALLS OR PIERS ARE SHOWN TO HAVE VERTICAL REINFORCING, FILL ALL CELLS CONTAINING VERTICAL REINFORCING WITH 3000 PSI PEA-GRAVEL CONCRETE OR GROUT CONFORMING WITH ASTM C476 WITH MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI, UNLESS NOTED OTHERWISE. LAY BLOCK A MAXIMUM OF 4'-0" HIGH PRIOR TO FILLING CELLS WITH CONCRETE. IF UNITS BECOME WET, DO NOT INSTALL UNITS UNTIL THEY ARE IN AN AIR-DRIED CONDITION.
10. PROVIDE LIGHT WEIGHT, HOLLOW, LOAD-BEARING CONCRETE MASONRY UNITS (CMU) CONFORMING TO ASTM C90, GRADE N-1, UNLESS NOTED OTHERWISE.
11. PROVIDE CONCRETE MASONRY WITH MINIMUM COMPRESSIVE STRENGTH, f'm = 1500 PSI CORRESPONDING TO UNIT STRENGTH OF 2000 PSI ON NET CROSS-SECTIONAL AREA OF CMU DETERMINED IN ACCORDANCE WITH ASTM C140.
12. SIDE AND TOP OF MASONRY WALL PANELS SHALL BE ANCHORED TO THE STRUCTURE AT A MAXIMUM SPACING OF 24" ON CENTER UNLESS NOTED OTHERWISE.
13. DO NOT PLACE CONDUIT OR PIPES IN THE SAME CELL AS REINFORCEMENT.

STRUCTURAL STEEL, STEEL JOISTS, STEEL DECK:

1. MATERIALS ARE TO BE AS FOLLOWS:

STRUCTURAL STEEL	(WIDE FLANGE SECTIONS) ASTM A992 (Fy = 50 ksi)
STRUCTURAL STEEL	(MISC STEEL) ASTM A36
STEEL PIPE ASTM	A501 OR ASTM A53
STRUCTURAL TUBING	ASTM A500, GRADE B (Fy = 46 ksi)
STRUCTURAL BOLTS	ASTM A325, MINIMUM 3/4" DIAMETER
ANCHOR RODS	ASTM F1554 GRADE 55/105 AS INDICATED
STUD CONNECTORS	(NELSON STUDS) 3/4" DIAMETER U.N.O.
ROOF DECK	1 1/2", 22 GA, TYPE B, BOTTOM PAINTED WHITE (80KSI STEEL)

2. PROVIDE TEMPORARY BRACING OF STRUCTURAL FRAMING UNTIL ALL PERMANENT BRACING, MOMENT CONNECTIONS AND FLOOR AND ROOF DECKS (DIAPHRAGMS) ARE COMPLETELY INSTALLED AND ALL TRUSS, TURNDOWN SLABS AND COLUMN/BEAM CONNECTIONS ARE COMPLETED.
3. THE STRUCTURAL DESIGN OF THE BUILDING IS BASED ON THE FULL INTERACTION OF ALL ITS COMPONENT PARTS, WITH NO PROVISIONS MADE FOR CONDITIONS OCCURRING DURING CONSTRUCTION. THEREFORE, CONTRACTOR SHALL PROVIDE ADEQUATE BRACING DURING CONSTRUCTION.
4. CONNECTIONS OF STRUCTURAL STEEL SHALL BE AS FOLLOWS, UNLESS NOTED OTHERWISE ON THE DRAWINGS:
   A. WHERE BEAM REACTIONS ARE NOT SHOWN ON THE DRAWINGS, CONNECTIONS SHALL BE DETAILED FOR THE MAXIMUM UNIFORM LOAD WHICH THE BEAM WILL SUPPORT (AS A SIMPLE BEAM) FOR THE SPAN SHOWN ON THE DRAWING. COMPONENTS AND CONNECTIONS ARE TO BE DESIGNED FOR 1.5 x THE MAXIMUM UNIFORM LOAD WHICH THE BEAM WILL SUPPORT AS A SIMPLE BEAM FOR THE SPAN SHOWN ON THE DRAWINGS.
   B. WHERE BEAM REACTIONS ARE SHOWN ON THE DRAWINGS, THE CONNECTIONS SHALL DEVELOP THE REACTIONS SHOWN. PROVIDE MINIMUM TWO 3/4" DIAMETER BOLTS AT CONNECTIONS. PROVIDE ERECTION BOLTS AT FIELD WELDED CONNECTIONS.
   C. WHERE REACTIONS ARE SUBJECT TO ECCENTRICITY, SUCH ECCENTRICITY SHALL BE TAKEN INTO ACCOUNT.
5. WELDING SHALL TO BE BY QUALIFIED WELDERS USING E70XX ELECTRODES AND IN ACCORDANCE WITH AWS D1.1.
6. STRUCTURAL STEEL SHALL BE DETAILED, FABRICATED AND ERECTED ACCORDING TO AISC LOAD AND RESISTANCE FACTOR DESIGN (LRFD) SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS.
7. SUBMIT SHOP DRAWINGS PREPARED IN ACCORDANCE WITH AISC MANUAL "DETAILING FOR STEEL CONSTRUCTION". DO NOT BEGIN FABRICATION UNTIL SHOP DRAWINGS ARE COMPLETED AND REVIEWED.
8. DO NOT USE GAS CUTTING TORCHES FOR CORRECTING FABRICATION ERRORS IN THE STRUCTURAL FRAMING.
9. SHEAR CONNECTORS: PROVIDE 3/4" DIAMETER SOLID FLUXED SHEAR CONNECTOR STUDS AUTOMATICALLY END WELDED THROUGH THE METAL DECK AS SHOWN ON THE DRAWINGS AND IN ACCORDANCE WITH THE RECOMMENDATIONS OF THE MANUFACTURER (NELSON DIVISION OF TRW OR APPROVED EQUAL).
10. STUD WELDING SHALL BE INSPECTED AND FIELD TESTED. STUDS FAILING THE TEST SHALL BE REPLACED AT THE CONTRACTORS EXPENSE.
11. ALL STRUCTURAL STEEL TO HAVE ONE (1) COAT OF STANDARD SHOP PRIMER. TOUCH UP ALL STRUCTURAL STEEL AFTER ERECTION WITH SAME PRIMER.
12. ALL EXPOSED STRUCTURAL STEEL WHICH RECEIVES ARCHITECTURAL PAINT SHOULD BE SHOP PRIMED. SEE ARCHITECTURAL DRAWINGS FOR LOCATION.
13. PAINT STRUCTURAL STEEL IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS. DO NOT PAINT FLAT SHEET SURFACES THAT ARE TO BE EMBEDDED IN CONCRETE. STEEL WHICH RECEIVES FIREPROOFING, CONNECTIONS DESIGNATED AS FRICTION TYPE, OR CONNECTIONS TO BE WELDED. STEEL SURFACES RECEIVING AUTOMATICALLY WELDED HEADED SHEAR CONNECTOR STUDS IN THE FIELD SHALL NOT BE PAINTED.
14. ALL THE EXPOSED STEEL SHOULD BE FABRICATED TO AISC'S ARCHITECTURALLY EXPOSED STRUCTURAL STEEL SPECIFICATIONS.
15. STRUCTURAL STEEL EXPOSED TO THE WEATHER SHALL BE GALVANIZED.
16. CONNECT THE METAL DECK TO STEEL SUPPORT WITH HILTI PINS AND HILTI SIDELAP FASTENERS.
17. FABRICATION AND ERECTION OF STEEL DECKING SHALL CONFORM TO THE LATEST EDITION OF THE STEEL DECK INSTITUTE DESIGN MANUAL FOR COMPOSITE DECKS, FORM DECKS AND ROOF DECKS AS APPLICABLE TO THE PROJECT.
18. PROVIDE STEEL DECKING CONTINUOUS OVER 3 SPANS MINIMUM WHERE SUPPORTING STRUCTURE PERMITS.
19. STEEL JOISTS AND JOIST GIRDERS SHALL BE FABRICATED AND ERECTED IN STRICT CONFORMANCE WITH THE LATEST EDITION OF "STANDARD SPECIFICATIONS AND LOAD TABLES FOR JOISTS AND JOIST GIRDERS", OF THE STEEL JOIST INSTITUTE (SJI).
20. JOISTS AND JOIST GIRDERS SHALL BE DESIGNED FOR THE UPLIFT WIND PRESSURE SHOWN ON THE ROOF PLAN IN ACCORDANCE WITH 1609.3 OF IBC 2018. ANY ADDITIONAL BRIDGING OR BRACING REQUIRED FOR BOTTOM CHORD SHALL BE SHOWN ON THE SHOP DRAWINGS. LOADS GIVEN FOR JOIST GIRDERS DO NOT INCLUDE WEIGHT OF JOIST GIRDERS.
21. BOLTED CONNECTIONS SHALL BE ASSEMBLED AND INSPECTED ACCORDING TO "SPECIFICATIONS FOR STRUCTURAL JOINTS USING ASTM A325 OR ASTM 490
22. WELDER'S CERTIFICATE SHALL BE AVAILABLE AT THE JOB SITE.

SLAB ON GRADE:

1. CONTROL JOINTS FOR SLAB ON GRADE SHALL BE LOCATED AS SHOWN ON PLAN, WITH A MAXIMUM JOINT SPACING OF 2 1/2 TIMES THE SLAB THICKNESS IN FEET. JOINTS SHALL BE FORMED USING SAW CUTS 1/8" WIDE (MAXIMUM) BY T/4 (1 1/4" MINIMUM) DEEP. SAW CUT JOINTS WITHIN 1-4 HOURS AFTER PLACING CONCRETE WITH EARLY ENTRY SAW AS PER ACI 360.
2. SIDEWALKS AND OTHER EXTERIOR SLABS ARE NOT SHOWN ON THE STRUCTURAL DRAWINGS. SEE ARCHITECTURAL, SITE AND CIVIL DRAWINGS FOR LOCATIONS, DIMENSIONS AND ELEVATIONS.
3. SEE ARCHITECTURAL DRAWINGS FOR EXACT LOCATION OF DEPRESSED SLAB AREAS AND DRAINS. FLOOR SLABS SHALL SLOPE TO ALL FLOOR DRAINS. GENERAL CONTRACTOR SHALL COORDINATE WITH PLUMBING DRAWINGS (TYPICAL).
4. USE EVAPORATION RETARDER ONE OR MORE TIMES AFTER THE STRIKEOFF WHEN HIGH TEMPERATURES, LOW HUMIDITY AND/OR WIND WILL CAUSE CRUSTING AND PLASTIC CRACKING.
5. SLAB WILL RECEIVE ONE COAT OF SONNEBORNE LAPIDOLTH FLOOR HARDENER. CURING COMPOUND SHOULD BE COMPATIBLE WITH HARDENER AND PLACED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.
6. FLOOR SLAB AREAS TO BE USED AS CASTING BED FOR TILT-UP WALL PANELS ARE TO RECEIVE TWO COATS OF A MEMBRANE FORMING BOND BREAKER (SPEC CHEM'S TILT WB OR EQUIVALENT), APPLIED AS RECOMMENDED BY MANUFACTURER. BOND BREAKER SHALL BE COMPATIBLE WITH ALL OTHER FLOOR TREATMENTS. IF FLOORS REQUIRE CLEANING AFTER WALLS ARE ERECTED, USE SPEC CHEM'S CITRUS CLEANER (OR EQUIVALENT APPLIED AS RECOMMENDED ARE ERECTED BY MANUFACTURER. AFTER CLEANING APPLY FINAL COAT OF SLAB HARDENER AND SEALER (SPEC CHEM'S SPEC HARD OR EQUIVALENT).

POST INSTALLED ANCHORS:

1. GENERAL
   A. PRE-CONSTRUCTION DUTIES OF THE CONTRACTOR:
      1. ARRANGE FOR AN ANCHOR MANUFACTURER'S REPRESENTATIVE TO PROVIDE ONSITE INSTALLATION TRAINING FOR ALL OF THEIR ANCHORING PRODUCTS SPECIFIED.
      2. PROVIDE THE STRUCTURAL ENGINEER OF RECORD MUST RECEIVE DOCUMENTED CONFIRMATION THAT ALL OF THE CONTRACTOR'S PERSONNEL WHO INSTALL ANCHORS ARE TRAINED PRIOR TO THE COMMENCEMENT OF INSTALLING ANCHORS.
   B. INSTALL POST-INSTALLED ANCHORS ONLY WHERE SPECIFIED ON THE CONSTRUCTION DOCUMENTS.
   C. OBTAIN APPROVAL FROM THE ENGINEER-OF-RECORD PRIOR TO INSTALLING POST-INSTALLED ANCHORS IN PLACE OF MISSING OR MISPLACED CAST-IN-PLACE ANCHORS.
   D. ANCHOR CAPACITY IS DEPENDANT UPON SPACING BETWEEN ADJACENT ANCHORS AND PROXIMITY OF ANCHORS TO EDGE OF CONCRETE. INSTALL ANCHORS IN ACCORDANCE WITH SPACING AND EDGE CLEARANCES INDICATED ON THE DRAWINGS OR INDICATED IN THE MANUFACTURER'S LITERATURE.
   E. EXISTING REINFORCING BARS AND OTHER EMBEDDED MATERIAL CONTAINED IN THE CONCRETE STRUCTURE MAY CONFLICT WITH SPECIFIC ANCHOR LOCATIONS.
      1. CONTRACTOR REVIEW THE EXISTING STRUCTURAL DRAWINGS AND UNDERTAKE TO LOCATE THE POSITION OF MATERIAL EMBEDDED IN THE CONCRETE AT THE LOCATIONS OF THE DETAILED ANCHORS, BY HILTI FERROSCAN, GPR, X-RAY, CHIPPING OR OTHER MEANS UNLESS IT IS NOTED ON THE DRAWINGS THAT THE BARS CAN BE CUT.
   F. DRILL AND CLEAN HOLES IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN INSTRUCTIONS.
   G. SUBMIT SUBSTITUTION REQUESTS, FOR PRODUCTS OTHER THAN THOSE SPECIFIED BELOW TO THE ENGINEER-OF-RECORD PRIOR TO USE ALONG WITH:
      1. PROVIDE CALCULATIONS THAT ARE PREPARED & SEALED BY A REGISTERED PROFESSIONAL ENGINEER.
      2. PROVIDE CALCULATIONS THAT DEMONSTRATE THE SUBSTITUTED PRODUCT IS CAPABLE OF ACHIEVING EQUIMELENT PERFORMANCE VALUES (MINIMUM) OF THE SPECIFIED PRODUCT USING THE APPROPRIATE DESIGN PROCEDURE AND/OR STANDARD(S) AS REQUIRED BY THE BUILDING CODE.
      3. INCLUDE CONSIDERATION OF CREEP, IN-SERVICE TEMPERATURE AND INSTALLATION TEMPERATURE IN SUBSTITUTION REQUEST.
      4. EVALUATION OF SUBSTITUTIONS WILL BE BASED ON THEIR HAVING AN ICC ESR SHOWING COMPLIANCE WITH THE RELEVANT BUILDING CODE FOR SEISMIC USES, LOAD RESISTANCE, INSTALLATION CATEGORY, AND AVAILABILITY OF COMPREHENSIVE INSTALLATION INSTRUCTIONS.
2. CONCRETE ANCHORS
   A. MEDIUM DUTY MECHANICAL AND SCREW ANCHORS FOR USE IN CRACKED AND UN-CRACKED CONCRETE THAT HAVE BEEN TESTED AND QUALIFIED FOR USE IN ACCORDANCE WITH ACI 355.2 AND ICC-ES AC193. PRE-APPROVED MECHANICAL AND SCREW ANCHORS INCLUDE:
      1. HILTI KWIK BOLT-TZ EXPANSION ANCHORS (ICC ESR-1917)
      2. HILTI KWIK HUS-EZ AND KWIK HUS EZ-I SCREW ANCHORS (ICC ESR-3027)
      3. SIMPSON STRONG-TIE "TITEN-HD" SCREW ANCHORS (ICC ESR-2713)
      4. SIMPSON STRONG-TIE "STRONG-BOLT 2" EXPANSION ANCHORS (ICC ESR-3037)
      5. DEWALT / POWERS POWER-STUD + SD2 EXPANSION ANCHORS (ICC ESR 2502)
   B. HEAVY DUTY MECHANICAL ANCHORS FOR CRACKED AND UN-CRACKED CONCRETE USE:
      1. HILTI HDA UNDERCUT ANCHORS (ICC ESR 1546)
      2. HILTI HSL-3 EXPANSION ANCHORS (ICC ESR 1545)
      3. DEWALT / POWERS ATOMIC + UNDERCUT ANCHOR (ICC ESR 3067)
   C. ADHESIVE ANCHORS FOR USE IN CRACKED AND UN-CRACKED CONCRETE THAT HAVE BEEN TESTED AND QUALIFIED FOR USE IN ACCORDANCE WITH ICC-ES AC308. PRE-APPROVED

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SHEET NAME: GENERAL SCHEDULES SHEET NUMBER: S-101 PROJECT NAME: PACE ROAD DISTRIBUTION CENTER - BUILDING 4

CONCRETE MIXTURE SCHEDULE

APPLICATION	EXPOSURE	F'c (AT 28-DAYS UNO)	MAXIMUM W/C	AIR CONTENT	NOMINAL MAXIMUM AGGREGATE SIZE (NOTE 4)	MAXIMUM CONCRETE WEIGHT
FOOTINGS	F0, S0, P0, C0	3000 PSI	SEE NOTE 2	SEE NOTE 3	1"	150 PCF
EXTERIOR SLAB-ON-GRADE (RAMPS)	F1, S0, P0, C1	4000 PSI AT 28-DAYS 4500 PSI AT 56-DAYS	0.45	4.5% ± 1.5%	1"	150 PCF
SLAB-ON-GRADE	F0, S0, P0, C0	4000 PSI	SEE NOTE 2	3% MAX	1 1/2"	150 PCF
LIGHTWEIGHT ELEVATED SLAB ON METAL DECK	F0, S0, P0, C0	4000 PSI	SEE NOTE 2	5.5% ± 1.5%	3/4"	150 PCF
TILT PANELS	F0, S0, P0, C1	SEE TILT PANEL ELEVATIONS	SEE NOTE 2	3% MAX	3/4"	150 PCF

CONCRETE MIXTURE:

1. EXPOSURE CATEGORIES AND CLASSES FOR SULFATES, PERMEABILITY, AND CORROSION PROTECTION OF REINFORCEMENT IS CLASS ZERO UNLESS NOTED OTHERWISE.
2. WHERE NO MAXIMUM WATER CEMENT RATIO IS NOTED FOR DURABILITY. PROPORTIONING OF WATER/CEMENT RATIO SHALL BE AS REQUIRED FOR SPECIFIED CONCRETE MIX DESIGN. WATER/CEMENT RATIO IS NOT APPLICABLE FOR DURABILITY REQUIREMENTS IN LIGHTWEIGHT CONCRETE.
3. WHERE AIR ENTRAINMENT IS NOT REQUIRED BY DESIGN, THE CONTRACTOR, INSTALLER, AND SUPPLIER MAY CHOOSE TO INCLUDE AIR ENTRAINMENT TO IMPROVE PLACEMENT AND FINISHING CHARACTERISTICS. AIR ENTRAINMENT IS NOT PERMITTED IN NORMAL WEIGHT CONCRETE TO RECEIVE A HARD TROWEL FINISH AND ENTRAPPED AIR SHALL NOT EXCEED 3%. AIR ENTRAINMENT IN LIGHTWEIGHT CONCRETE SLABS IS REQUIRED TO MEET FIRE RATING REQUIREMENTS. SLABS SHALL BE PROPERLY FINISHED TO AVOID SURFACE IMPERFECTIONS, SUCH AS BLISTERING OR DELAMINATION.
4. COARSE AGGREGATE SHALL BE ASTM C 33, GRADED. SELECT GRADING CLASS PER TYPE OF CONSTRUCTION OR LOCATION USED, AND IN RELATION TO SPECIFIC WEATHERING REGION. AGGREGATE SHALL BE FROM A SINGLE SOURCE. #67 GRADING SHALL BE USED FOR CONCRETE WITH 3/4 INCH MAXIMUM: #67 GRADING SHALL BE USED FOR CONCRETE WITH 1 INCH MAXIMUM: A WELL BLENDED MIX OF #4, #57 AND #89 (1 1/2" TI 3/8" NOMINAL SIZE) SHALL BE USED FOR CONCRETE WITH 1 1/2 INCH MAXIMUM.
5. FINE AGGREGATE FOR INTERIOR CONCRETE SLAB SHALL CONSIST OF A MINIMUM 70% NATURAL SAND.
6. MIX DESIGN SUBMITTAL FOR CONCRETE WITH BLENDED AGGREGATES SHALL INCLUDE AGGREGATE GRADING AND VOID CONTENT OF THE COMBINED AGGREGATE. IT IS ACCEPTABLE TO USE A DIFFERENT BLEND OF AGGREGATES WITH 1 1/2 INCH MAXIMUM BY DOING ONE OF THE FOLLOWING:
   - SUBMIT A MIX ANALYSIS WITH A COARSENESS FACTOR CHART SHOWING THE BLEND FALLS WITHIN THE "OPTIMAL" AREA OF THE CHART. REFER TO ACI 302.1R-15 - CHAPTER 8.
   - SUBMIT DOCUMENTATION FOR THE SHRINKAGE POTENTIAL OF THE CONCRETE MIXTURE. LENGTH CHANGE OF CONCRETE, DETERMINED BY ASTM C 157, WITH 7 DAYS OF MOIST CURING FOLLOWED BY 21 DAYS OF DRYING SHALL NOT EXCEED 0.05%.
   - PLACE A TRIAL SLAB.
7. FLY ASH IS NOT PERMITTED IN CONCRETE SLAB OR TILT-UP WALL PANEL MIX DESIGNS.

CONCRETE PROTECTION FOR REINFORCEMENT

F'c =			CONCRETE COVER (INCHES)
1. CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH.			3"
2. CONCRETE EXPOSED TO EARTH OR WEATHER.		#6 THROUGH #18 BARS	2"
		#5 BARS, W31 OR D31 WIRE, & SMALLER	1 1/2"
		COLUMN TIES	1 1/2"
		PT SLABS	1"
3. CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH GROUND.	SLABS, WALLS & JOISTS	#14 & #18 BARS	1 1/2"
		#11 & SMALLER	3/4"
	BEAMS & COLUMNS	PRIMARY REINFORCEMENT	1 1/2"
		TIES, STIRRUPS, & SPIRALS	1"
	PT SLAB		3/4" (1 3/4" @ UNRESTRAINED AREAS)

LAP CLASS A/CLASS B SCHEDULE

F'c =	3000 PSI				4000 PSI				5000 PSI				6000 PSI				7000 PSI				8000 PSI			
	LENGTH				LENGTH				LENGTH				LENGTH				LENGTH				LENGTH			
BAR SIZE	TOP BARS		OTHER BARS		TOP BARS		OTHER BARS		TOP BARS		OTHER BARS		TOP BARS		OTHER BARS		TOP BARS		OTHER BARS		TOP BARS		OTHER BARS	
LAP CLASS	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
#3	16"	17"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"
#4	18"	23"	16"	18"	16"	20"	16"	16"	16"	18"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"	16"
#5	26"	34"	20"	26"	23"	29"	17"	23"	20"	26"	16"	20"	18"	24"	16"	18"	17"	22"	16"	17"	16"	21"	16"	16"
#6	35"	46"	27"	35"	31"	40"	24"	31"	28"	36"	21"	28"	25"	33"	20"	25"	23"	30"	18"	23"	22"	28"	17"	22"
#7	57"	74"	44"	57"	50"	65"	38"	50"	45"	58"	34"	45"	41"	53"	31"	41"	38"	49"	29"	38"	35"	46"	27"	35"
#8	72"	93"	55"	72"	62"	81"	48"	62"	56"	72"	43"	56"	51"	66"	39"	51"	47"	61"	36"	47"	44"	57"	34"	44"
#9	87"	113"	67"	87"	76"	98"	58"	76"	68"	88"	52"	68"	62"	80"	48"	62"	57"	74"	44"	57"	54"	70"	41"	54"
#10	106"	137"	82"	106"	92"	119"	71"	92"	82"	107"	63"	82"	75"	97"	58"	75"	69"	90"	54"	69"	65"	84"	50"	65"

EXISTING DRAWINGS FOR REFERENCE ONLY

ABBREVIATIONS SCHEDULE

ABBREV	WORD
#	NUMBER/POUND
&	AND
@	AT
ALT	ALTERNATE
ANC	ANCHOR
ANCB	ANCHOR BOLT
APPROX	APPROXIMATE
ARCH	ARCHITECT/ARCHITECTURAL
BB	BOND BEAM
BF	BRACE FRAME
BLDG	BUILDING
BLK	BLOCK
BM	BEAM
BO	BOTTOM OF
BOD	BOTTOM OF DECK
BOS	BOTTOM OF STEEL
BOT	BOTTOM
BP	BASEPLATE
BRGP	BEARING PLATE
BRK	BRICK
BSMT	BASEMENT
BT	BENT
BTWN	BETWEEN
BW	BOTHWAYS
BYD	BEYOND
CANT	CANTILEVER
CANT	CANTILEVER
CC	CLEAR COVER
CFD	COMPOSITE FORM DECK
CH	CHANNEL
CIP	CAST-IN-PLACE
CJ	CONTROL JOINT
CL	CENTERLINE
CLR	CLEAR
CMD	COMPOSITE METAL DECK
CMU	CONCRETE MASONRY UNIT
CNR	CORNER
CNSTR	CONSTRUCTION
COL	COLUMN
CONC	COLUMN
CONJ	CONSTRUCTION JOINT
CONN	CONNECTION
CONT	CONTINUOUS
CONTR	CONTRACTOR
COORD	COORDINATE
CR	CENTER REINFORCING
DBL	DOUBLE
DEG	DEGREE
DEMO	DEMOLITION
DIA	DIAMETER
DIAG	DIAGONAL
DIAPH	DIAPHRAGM
DIM	DIMENSION
DL	DEAD LOAD
DN	DOWN
DTL	DETAIL(S)
DWG	DRAWING
EA	EACH
EBA	EPOXY BONDED ANCHOR
EE	EACH END
EF	EACH FACE
EJ	EXPANSION JOINT
ELEV	ELEVATION
EMBED	EMBEDMENT/EMBEDDED
EOD	EDGE OF DECK
EOS	EDGE OF SLAB
EPCT	EPOXY COAT
EQ	EQUAL
EQUIP	EQUIPMENT
ES	EACH SIDE
EW	EACH WAY
EXIST	EXISTING
EXP	EXPANSION
EXPANC	EXPANSION ANCHOR(S)
EXT	EXTERIOR
FAS	FASTENER(S)
FD	FLOOR DRAIN
FDN	FOUNDATION
FF	FINISH FLOOR
FIN	FINISH
FLR	FLOOR
FO	FACE OF
FOC	FACE OF CONCRETE
FOS	FACE OF STEEL
FS	FAR SIDE
FT	FOOT OR FEET
FTG	FOOTING
FTR	FUTURE
FV	FIELD VERIFY
GA	GAGE
GALV	GALVANIZED
GEN	GENERAL
GL	GRIDLINE
GN	GENERAL NOTES
GR	GRADE
GS	GRIND SMOOTH
HC	HOLLOW CORE
HCA	HEADED CONCRETE ANCHOR
HDG	HOT DIPPED GALVANIZED
HI	HIGH
HK	HOOK
HORIZ	HORIZONTAL
HSS	HOLLOW STRUCTURAL STEEL
HT	HEIGHT

ABBREVIATIONS SCHEDULE

ABBREV	WORD
ID	INSIDE DIAMETER
IN	INCH
INT	INTERIOR
INV	INVERT/INVERTED
ISF	INSIDE FACE
JST	JOIST
JST-GRD	JOIST-GIRDER
JT	JOINT
K	KIP (THOUSAND POUNDS)
KSI	KIPS PER SQUARE INCH
L	ANGLE
LAT	LATERAL
LBS	POUND(S)
LF	LATERAL FRAME
LFT	LINEAR FOOT
LGMF	LIGHT GAGE METAL FRAMING
LLH	LONG LEG HORIZONTAL
LLV	LONG LEG VERTICAL
LO	LOW
LSH	LONG SLOTTED HOLES
LVS	LONG VERTICAL SLOT
M	MATERIAL
MAS	MASONRY
MAX	MAXIMUM
MECH	MECHANICAL
MEP	MECHANICAL, ELECTRICAL, PLUMBING
MEZZ	MEZZANINE
MFR	MANUFACTURER
MID	MIDDLE
MIN	MINIMUM
MISC	MISCELLANEOUS
ML	MASONRY LINTEL
MO	MASONRY OPENING
MOM	MOMENT
MW	MASONRY WALL
N/A	NOT APPLICABLE
NOM	NOMINAL
NS	NEAR SIDE
NTS	NOT TO SCALE
OC	ON CENTER
OD	OUTSIDE DIAMETER
OH	OPPOSITE HAND
OPNG	OPENING
OPP	OPPOSITE
OSF	OUTSIDE FACE
PC	PRECAST
PCF	POUNDS PER CUBIC FOOT
PED	PEDESTAL
PEMB	PRE-ENGINEERED METAL BUILDING
PERP	PERPENDICULAR
PL	PLATE
PREFAB	PREFABRICATED
PROJ	PROJECTION
PSF	POUND PER SQUARE FOOT
PSI	POUND PER SQUARE INCH
PT	POINT
R	RISER
RAD	RADIUS
RD	ROOF DRAIN
REBAR	REINFORCING BAR
REF	REFERENCE
REINF	REINFORCING/REINFORCED
REQ	REQUIRED
REQT	REQUIREMENT(S)
RTU	ROOF TOP UNIT
SCHED	SCHEDULE
SECT	SECTION
SHT	SHEATHING
SIM	SIMILAR
SL	SLOPE
SOG	SLAB ON GRADE
SPA	SPACES/SPACING
SPEC	SPECIFICATION(S)
SQ	SQUARE
SS	STAINLESS STEEL
ST	STRUCTURAL TUBE
STD	STANDARD
STIFF	STIFFENER
STL	STEEL
STRUC	STRUCTURAL
SYM	SYMMETRICAL
T&B	TOP AND BOTTOM
TD	TURN DOWN
THD	THREAD(S)
THK	THICK/THICKNESS
TOB	TOP OF BEAM
TOC	TOP OF CONCRETE
TOF	TOP OF FOUNDATION/FOOTING
TOG	TOP OF GRADE
TOJ	TOP OF JOIST
TOL	TOP OF LEDGE
TOPED	TOP OF PEDESTAL
TOS	TOP OF STEEL
TOW	TOP OF WALL
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
UT	UTILITIES
VERT	VERTICAL
W/	WITH
W/O	WITHOUT
WF	WIDE FLANGE BEAM
WP	WORK POINT
WS	WATERSTOP
WT	WEIGHT
WWF	WELDED WIRE FABRIC

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Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

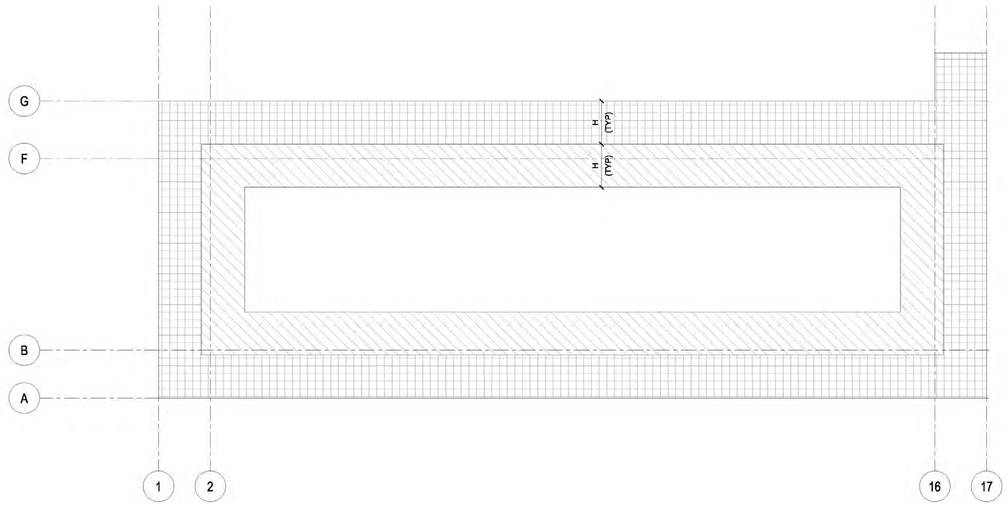
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Sheet Title
GENERAL SCHEDULES

Sheet No.
S-101

CONSTRUCTION SET

9.107

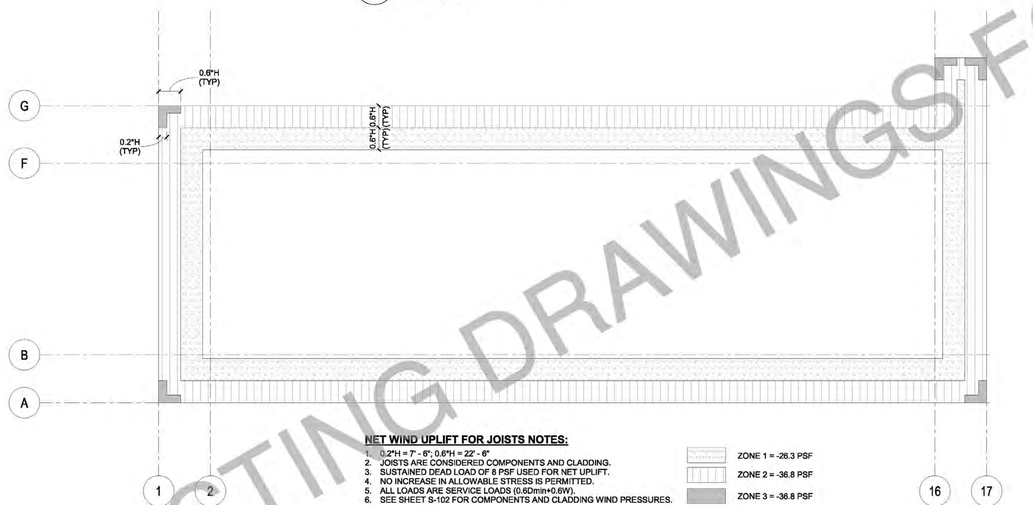


NET WIND UPLIFT FOR GIRDERS NOTES:

1. H = 37' - 6"
2. GIRDERS ARE PART OF MAIN WIND FORCE RESISTING SYSTEM.
3. SUSTAINED DEAD LOAD OF 8 PSF USED FOR NET UPLIFT.
4. NO INCREASE IN ALLOWABLE STRESS IS PERMITTED.
5. ALL LOADS ARE SERVICE LOADS (0.6Dmin+0.6W).
6. SEE SHEET S-101 FOR MWFRS WIND PRESSURES.

ZONE 1 & ZONE 2 = -20.1 PSF
ZONE 3 = -11.1 PSF
ZONE 4 = -8.7 PSF

2 / S-102 **NET WIND UPLIFT FOR GIRDERS PLAN**
SCALE: 1/64" = 1'-0"



NET WIND UPLIFT FOR JOISTS NOTES:

1. 0.2*H = 7' - 6"; 0.6*H = 22' - 6"
2. JOISTS ARE CONSIDERED COMPONENTS AND CLADDING.
3. SUSTAINED DEAD LOAD OF 8 PSF USED FOR NET UPLIFT.
4. NO INCREASE IN ALLOWABLE STRESS IS PERMITTED.
5. ALL LOADS ARE SERVICE LOADS (0.6Dmin+0.6W).
6. SEE SHEET S-102 FOR COMPONENTS AND CLADDING WIND PRESSURES.

ZONE 1 = -26.3 PSF
ZONE 2 = -36.8 PSF
ZONE 3 = -36.8 PSF
ZONE 1' = -11.5 PSF

3 / S-102 **NET WIND UPLIFT FOR JOISTS PLAN**
SCALE: 1/64" = 1'-0"

GRAVITY LOADS:

SLAB ON GRADE LIVE LOAD	250 PSF
ROOF DEAD LOAD	
ROOF MEMBRANE AND INSULATION	2 PSF
METAL DECK	2 PSF
JOIST FRAMING	3 PSF
SPRINKLERS	3 PSF
DUCTS, LIGHTING, MISC	3 PSF
TOTAL ROOF DEAD LOAD	13 PSF
ROOF LIVE LOAD (REDUCIBLE)	20 PSF
DOWNWARD (POSITIVE) C&C WIND LOAD	16.7 PSF (ULTIMATE)

JOIST LOAD NOTES:
1. ALL LOADS SHOWN ARE UNFACTORED
2. ALL CODE LOAD COMBINATIONS AND LIMIT STATES SHOULD BE ACCOUNTED FOR BY JOIST DESIGNER

SEISMIC LOADS

ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE PROCEDURE FOR SEISMIC DESIGN OF BUILDINGS - FBC 2023/ASCE 7-22 SECTION 12.8

MAXIMUM GROUND MOTION 0.2 sec response	Ss = 0.075g
MAXIMUM GROUND MOTION 1.0 sec response	S1 = 0.032g
SHORT PERIOD MAPPED SPECTRAL RESPONSE ACCELERATION	Sms = 0.1g
1.0 SECOND PERIOD MAPPED SPECTRAL RESPONSE ACCELERATION	Sm1 = 0.068g
SHORT PERIOD SPECTRAL RESPONSE COEFFICIENT	Sds = 0.067g
1.0 SECOND PERIOD SPECTRAL RESPONSE COEFFICIENT	Sd1 = 0.046g
RISK CATEGORY	II
SEISMIC IMPORTANCE FACTOR, Ie	1.0
SITE CLASS	D
SEISMIC DESIGN CATEGORY	A

WIND LOADS:

ANALYSIS PROCEDURE:

ANALYTICAL METHOD - ASCE 7-22 DIRECTIONAL METHOD	
ULTIMATE WIND SPEED (3-SECOND GUST)	140 MPH (FIG. 1609.3 (1) - FBC 2023)
NOMINAL WIND SPEED	108.4 MPH (T.1609.3.1 - FBC 2023)
RISK CATEGORY	II (TABLE 1604.5 - FBC 2023)
WIND EXPOSURE	C (SECT. 1609.4.3 - FBC 2023)

DESIGN FOR MAIN WIND-FORCE RESISTING SYSTEM (MWFRS) AND COMPONENTS AND CLADDING (C & C) BASED ON ULTIMATE WIND SPEED Vult = 140 MPH. WIND SPEED CONVERSION FOR NOMINAL DESIGN WIND SPEED BASED ON EQUATION 16-17 (FBC 2023).

Vasd = Vult x 0.6

INTERNAL PRESSURE COEFFICIENT	+0.18/-0.18 (ENCLOSED)

COEFFICIENTS AND PRESSURES (LRFD LOADS):

VELOCITY PRESSURE @ HEIGHT z=H, Qh	44.5 PSF
VELOCITY PRESSURE @ HEIGHT z, Qz	44.5 PSF
GUST FACTOR, G	0.85
TOPOGRAPHIC FACTOR, Kzt	1.00 (FIG 26.8-1 ASCE)
WIND DIRECTIONALITY FACTOR, Kd	0.85 (TABLE 26.6-1 ASCE)
VELOCITY PRESSURE EXPOSURE COEF. @ HEIGHT z=H, Kh	1.04
VELOCITY PRESSURE EXPOSURE COEF. @ HEIGHT z, Kz	1.04

MAIN WIND FORCE RESISTING SYSTEM (ULTIMATE):

WALL (MWFRS - WINDWARD)	+38.7 PSF
WALL (MWFRS - LEEWARD)	-26.9 PSF
WALL (MWFRS - SIDE)	-34.5 PSF
ROOF (MWFRS - ZONE1*)	-42.1/+8.0 PSF
ROOF (MWFRS - ZONE2*)	-42.1/+8.0 PSF
ROOF (MWFRS - ZONE3*)	-26.9/+8.0 PSF
ROOF (MWFRS - ZONE4*)	-19.4/+8.0 PSF

*ROOF ZONE1 IS APPLIED FOR HORIZONTAL DISTANCE OF 0 TO H/2 FROM WINDWARD EDGE
*ROOF ZONE2 IS APPLIED FOR HORIZONTAL DISTANCE OF H/2 TO H FROM WINDWARD EDGE
*ROOF ZONE3 IS APPLIED FOR HORIZONTAL DISTANCE OF H TO 2H FROM WINDWARD EDGE
*ROOF ZONE4 IS APPLIED FOR HORIZONTAL DISTANCE OF > 2H FROM WINDWARD EDGE

BUILDING HAS BEEN DESIGNED FOR WIND LOADS ASSUMING THE BUILDING MEETS ASCE-7 2022 DEFINITION OF AN "ENCLOSED" BUILDING. THIS REQUIRES THAT ALL EXTERIOR DOOR REMAIN CLOSED EXCEPT DURING LOADING/UNLOADING. ALL EXTERIOR DOORS SHALL REMAIN CLOSED IN THE EVENT OF A HIGH WIND EVENT (WINDS EXCEEDING 40 MPH OR EXPECTED TO EXCEED 40 MPH, SERVICE LEVEL)

POSITIVE/NEGATIVE WIND PRESSURE ON GLAZING AND WALL COMPONENT (SERVICE)

BUILDING	LOCATION ON BUILDING	WIND PRESSURE			
		<20 FT²	<50 FT²	<100 FT²	<200 FT²
MAIN	FIELD [4]	-30.9/+28.5 PSF	-29.6/+27.2 PSF	-27.9/+25.6 PSF	-26.7/+24.3 PSF
	CORNER [5]	-38.0/+28.5 PSF	-35.5/+27.2 PSF	-32.1/+25.6 PSF	-29.6/+24.3 PSF

ROOF WIND UPLIFT/DOWNWARD PRESSURES (SERVICE)

BUILDING	LOCATION ON BUILDING	WIND PRESSURE			
		<20 FT²	<50 FT²	<100 FT²	<200 FT²
MAIN	[1]	-49.6/+12.7 PSF	-46.3/+11.9 PSF	-42.0/+10.9 PSF	-38.7/+10.1 PSF
	[2]	-65.4/+12.7 PSF	-61.2/+11.9 PSF	-55.6/+10.9 PSF	-51.4/+10.1 PSF
	[3]	-89.1/+12.7 PSF	-81.6/+11.9 PSF	-70.5/+10.9 PSF	-62.1/+10.1 PSF
	[1*]	-28.5/+12.7 PSF	-28.5/+11.9 PSF	-28.5/+10.9 PSF	-28.5/+10.1 PSF

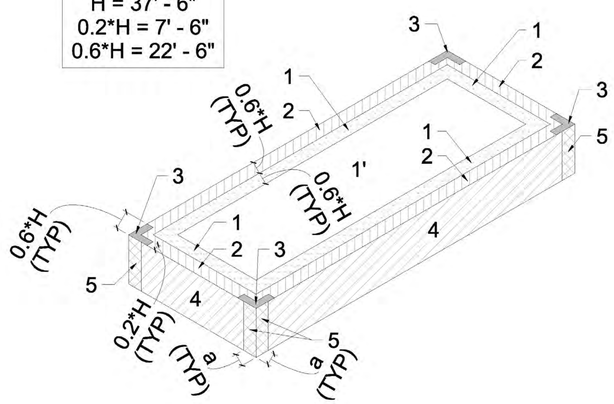
POSITIVE/NEGATIVE WIND PRESSURE ON GLAZING AND WALL COMPONENT (ULTIMATE)

BUILDING	LOCATION ON BUILDING	WIND PRESSURE			
		<20 FT²	<50 FT²	<100 FT²	<200 FT²
MAIN	FIELD [4]	-51.4/+47.5 PSF	-49.3/+45.4 PSF	-46.5/+42.6 PSF	-44.4/+40.5 PSF
	CORNER [5]	-63.3/+47.5 PSF	-59.1/+45.4 PSF	-53.5/+42.6 PSF	-49.3/+40.5 PSF

ROOF WIND UPLIFT/DOWNWARD PRESSURES (ULTIMATE)

BUILDING	LOCATION ON BUILDING	WIND PRESSURE			
		<20 FT²	<50 FT²	<100 FT²	<200 FT²
MAIN	[1]	-82.6/+21.1 PSF	-77.1/+19.8 PSF	-69.9/+18.1 PSF	-64.5/+16.7 PSF
	[2]	-108.9/+21.1 PSF	-101.9/+19.8 PSF	-92.7/+18.1 PSF	-85.7/+16.7 PSF
	[3]	-148.4/+21.1 PSF	-136.0/+19.8 PSF	-117.5/+18.1 PSF	-103.5/+16.7 PSF
	[1*]	-47.5/+21.1 PSF	-47.5/+19.8 PSF	-47.5/+18.1 PSF	-47.5/+16.7 PSF

a = 15' - 0"
H = 37' - 6"
0.2*H = 7' - 6"
0.6*H = 22' - 6"



THIS BUILDING IS NOT DESIGNED TO MEET REQUIREMENTS OF FM-GLOBAL

EXISTING DRAWINGS FOR REFERENCE ONLY

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PACE ROAD DISTRIBUTION CENTER - BUILDING 4

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PACE RD & PEARCE RD
AUBURNDALE, FL 33823

Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
GRAVITY & LATERAL LOADING

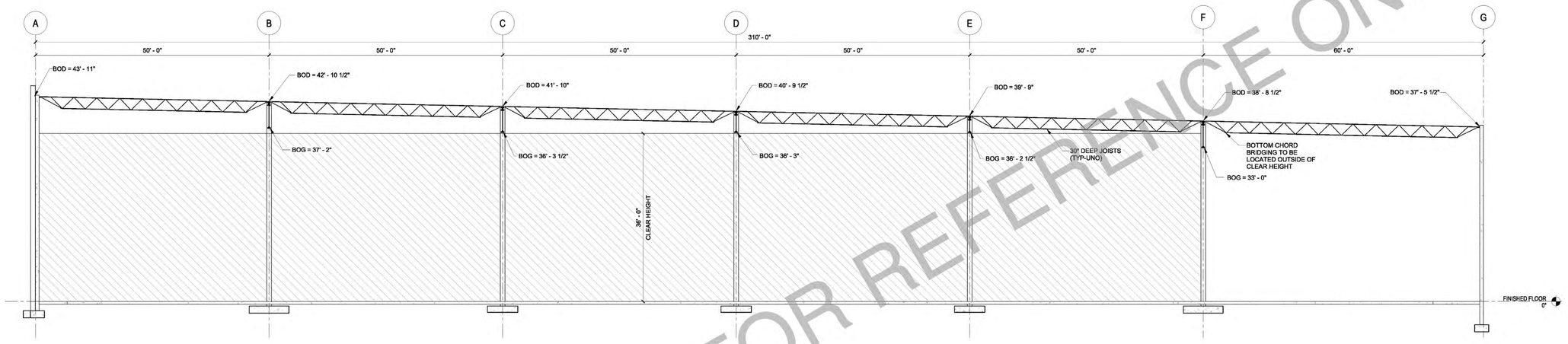
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S-102

CONSTRUCTION SET

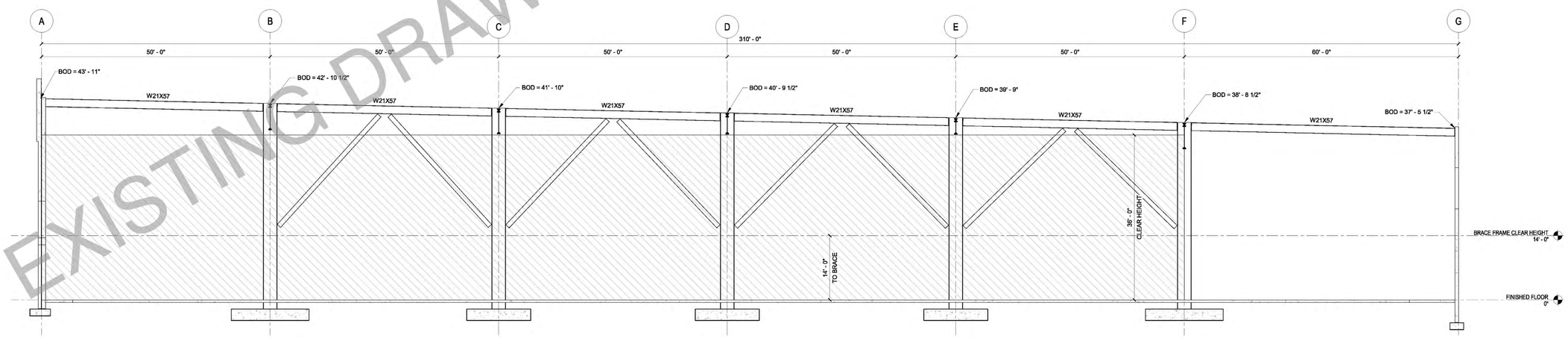
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FILENAME: C:\Users\BjanGhanagozlou\Documents\24.03.058-Nelson-Pace Road DC-BLDG 4-R23_bjghanagozlouFRTGU.rvt

EXISTING DRAWINGS FOR REFERENCE ONLY



1 S-103 **BUILDING CLEAR HEIGHT AT JOISTS ELEVATION**
SCALE: 1" = 10'-0"



2 S-103 **BUILDING CLEAR HEIGHT AT BRACE FRAMES ELEVATION**
SCALE: 1" = 10'-0"

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Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
BUILDING CLEAR HEIGHT ELEVATIONS

Sheet No.
S-103

CONSTRUCTION SET

9.109

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SHEET NAME: SPECIAL INSPECTIONS SHEET NUMBER: S-104 PROJECT NAME: PACE ROAD DISTRIBUTION CENTER - BUILDING 4

SCHEDULE OF SPECIAL INSPECTION SERVICES

PROJECT

MATERIAL / ACTIVITY	SERVICE	APPLICABLE TO THIS PROJECT Y/N	EXTENT	AGENT*	DATE COMPLETED
1704.2 Inspection of Fabricators					
Verify fabrication/quality control procedures.	In-plant review (3)	Y	Periodic	1	.
1705.1.1 Special Cases (work unusual in nature, including but not limited to alternative materials and systems, unusual design applications, materials and systems with special manufacturer's requirements	Submittal review, shop (3) and/or field inspection	N		.	.
1705.2 Steel Construction					
1. Fabricator and erector documents (Verify reports and certificates as listed in AISC 360, chapter N, paragraph 3.2 for compliance with construction documents)	Submittal Review	Y	Each submittal	1	.
2. Material verification of structural steel	Shop (3) and field inspection	Y	Periodic	1	.
3. Embedments (Verify diameter, grade, type, length, embedment. See 1705.3 for anchors)	Field inspection	Y	Periodic	1	.
4. Verify member locations, braces, stiffeners, and application of joint details at each connection comply with construction documents	Field inspection	Y	Periodic	1	.
5. Structural steel welding:					
a. Inspection tasks Prior to Welding (Observe, or perform for each welded joint or member, the QA tasks listed in AISC 360, Table N5.4-1)	Shop (3) and field inspection	Y	Observe or perform as noted (4)	1	.
b. Inspection tasks During Welding (Observe, or perform for each welded joint or member, the QA tasks listed in AISC 360, Table N5.4-2)	Shop (3) and field inspection	Y	Observe (4)	1	.
c. Inspection tasks After Welding (Observe, or perform for each welded joint or member, the QA tasks listed in AISC 360, Table N5.4-3)	Shop (3) and field inspection	Y	Observe or perform as noted (4)	1	.
d. Nondestructive testing (NDT) of welded joints: see Commentary					
1) Complete penetration groove welds 5/16" or greater in risk category III or IV	Shop (3) or field Ultrasonic testing - 100%	N	Periodic	.	.
2) Complete penetration groove welds 5/16" or greater in risk category II	Shop (3) or field Ultrasonic testing - 10% of welds mimimum	N	Periodic	.	.
3) Thermally cut surfaces of access holes when material t > 2"	Shop (3) or field magnetic Partical or Penetrant testing	N	Periodic	.	.
4) Welded joints subject to fatigue when required by AISC 360, Appendix 3, Table A-3.1	Shop (3) or field radiographic or Ultrasonic testing	N	Periodic	.	.
5) Fabricator's NDT reports when fabricator performs NDT	Verify reports	N	Each Submittal (5)	.	.
6. Structural steel bolting:	Shop (3) and field inspection				
a. Inspection tasks Prior to Bolting (Observe, or perform for each bolted connection, in accordance with QA tasks listed in AISC 360, Table N5.6-1)		Y	Observe or perform as noted (4)	1	.
b. Inspection tasks During Bolting (Observe the QA tasks listed in AISC 360, Table N5.6-2)		Y	Observe (4)	1	.
1) Pre-tensioned and slip-critical joints		N			
a) Turn-of-nut with matching markings		N	Periodic	1	.
b) Direct tension indicator		N	Periodic	.	.
c) Twist-off type tension control bolt		N	Periodic	.	.
d) Turn-of-nut without matching markings		N	Continuous	.	.
e) Calibrated wrench		N	Continuous	.	.
2) Snug-tight joints		Y	Periodic	1	.
c. Inspection tasks After Bolting (Perform tasks for each bolted connection in accordance with QA tasks listed in AISC 360, Table N5.6-3)		Y	Perform (4)	1	.
7. Inspection of steel elements of composite construction prior to concrete placement in accordance with QA tasks listed in AISC 360, Table N6.1	Shop (3) and field inspection and testing	N	Observe or perform as noted (4)	.	.
1705.2.2 Steel Construction Other Than Structural Steel					
1. Material verification of cold-formed steel deck:					
a. Identification markings	Field inspection	Y	Periodic	1	.
b. Manufacturer's certified test reports	Submittal review	Y	Each Submittal	1	.
2. Connection of cold-formed steel deck to supporting structure:	Shop (3) and Field inspection				
a. Welding		N	Periodic	.	.
b. Other fasteners (in accordance with AISC 360, Section N6)					.
1) Verify fasteners are in conformance with approved submittal		Y	Periodic	1	.
2) Verify fasteners installation is in conformance with approved submittal and manufacturer's recommendations		Y	Periodic	1	.
3. Reinforcing steel	Shop (3) and field inspection				
a. Verification of weldability of steel other than ASTM A706		N	Periodic	.	.
b. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, boundary elements of special concrete structural walls and shear reinforcement		N	Continuous	.	.
c. Shear reinforcement		N	Continuous	.	.
d. Other reinforcing steel		N	Periodic	.	.
1705.3 Concrete Construction					
1. Inspection of reinforcing steel installment (see 1705.2.2 for welding)	Shop (3) and field inspection	Y	Periodic	1	.
2. Inspection of prestressing steel installation	Shop (3) and field inspection	N	Periodic	.	.
3. Inspection of anchors cast in concrete where allowable loads have been increased per section 1908.5 or where strength design is used	Shop (3) and field inspection	N	Periodic	.	.

SCHEDULE OF SPECIAL INSPECTION SERVICES

PROJECT

MATERIAL / ACTIVITY	SERVICE	APPLICABLE TO THIS PROJECT Y/N	EXTENT	AGENT*	DATE COMPLETED
4. Inspection of anchors and reinforcing steel post-installed in hardened concrete: Per research reports including verification of anchor type, anchor dimensions, hole dimensions, hole cleaning procedures, anchor spacing, edge distances,concrete embedment and tightening torque	Field inspection	Y	Periodic or as required by the research report issued by an approved source	1	.
5. Verify use of approved design mix	Shop (3) and field inspection	Y	Periodic	1	.
6. Fresh concrete sampling, perform slump and air content tests and determine temperature of concrete	Shop (3) and field inspection	Y	Continuous	1	.
7. Inspection for concrete and shotcrete placement for proper application techniques	Shop (3) and field inspection	N	Continuous	.	.
8. Inspection for maintenance of specified curing temperature and techniques	Shop (3) and field inspection	Y	Periodic	1	.
9. Inspection of prestressed concrete:	Shop (3) and field inspection	N		.	.
a. Application of prestressing force		N	Continuous	.	.
b. Grouting of bonded prestressing tendons in seismic-force-resisting system		N	Continuous	.	.
10. Erection of precast concrete members					
a. Inspect in accordance with construction documents	Field inspection	Y	In Accordance with Construction Documents	.	.
b. Perform inspections of welding and bolting in accordance with Section 1705.2	Field inspection	Y	In Accordance with Section 1705.2	.	.
11. Verification of in-situ concrete strength, prior to stressing of tendons in post tensioned concrete and prior to removal of shores and forms from beams and structural slabs	Review field testing and laboratory reports	N	Periodic		.
12. Inspection of formwork for shape, lines, location and dimensions	Field inspection	Y	Periodic	1	.
13. Concrete strength testing and verification of compliance with construction documents	Field testing and review of laboratory reports	Y	Periodic	1	.
1705.4 Masonry Construction					
(A) Level A, B and C Quality Assurance:					
1. Verify compliance with approved submittals	Field inspection	Y	Periodic	1	.
(B) Level B Quality Assurance:					
1. Verification of f'm and f_{AAC} prior to construction	Testing by unit strength method or prism test method	Y	Periodic	1	.
(C) Level C Quality Assurance:					
1. Verification of f'm and fAAC prior to construction and for every 5,000 SF during construction	Testing by unit strength method or prism test method	N	Periodic	.	.
2. Verification of proportions of materials in premixed or preblended mortar, prestressing grout, and grout other than self-consolidating grout, as delivered to the project site	Field inspection	N	Continuous	.	.
3. Verify placement of masonry units	Field inspection	N	Periodic	.	.
(D) Levels B and C Quality Assurance:					
1. Verification of Slump Flow and Visual Stability Index (VSI) of self-consolidating grout as delivered to the project	Field testing	N	Continuous	.	.
2. Verify compliance with approved submittals	Field inspection	Y	Periodic	1	.
3. Verify proportions of site-mixed mortar, grout and prestressing grout for bonded tendons	Field inspection	Y	Periodic	1	.
4. Verify grade, type, and size of reinforcement and anchor bolts, and prestressing tendons and anchorages	Field inspection	Y	Periodic	1	.
5. Verify construction of mortar joints	Field inspection	Y	Periodic	1	.
6. Verify placement of reinforcement, connectors, and prestressing tendons and anchorages	Field inspection	Y	Level B - Periodic	1	.
		N	Level C - Continuous	.	.
7. Verify grout space prior to grouting	Field inspection	Y	Level B - Periodic	1	.
		N	Level C - Continuous	.	.
8. Verify placement of grout and prestressing grout for bonded tendons	Field inspection	Y	Continuous	1	.
9. Verify size and location of structural masonry elements	Field inspection	Y	Periodic	1	.
10. Verify type, size, and location of anchors, including details of anchorage of masonry to structural members, frames, or other construction	Field inspection	Y	Level B - Periodic	1	.
		N	Level C - Continuous	.	.
11. Verify welding of reinforcement (see 1705.2.2)	Field inspection		Continuous	.	.
12. Verify preparation, construction, and protestion of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F)	Field inspection	Y	Periodic	1	.
13. Verify application and measurement of prestressing force	Field inspection	N	Continuous	.	.
14. Verify placement of AAC masonry units and construction of thin-bed mortar joints (first 5000 SF of AAC masonry)	Field inspection	N	Continuous	.	.
15. Verify placement of AAC masonry units and construction of thin-bed mortar joints (after the first 5000 SF of AAC masonry)	Field inspection	N	Level B - Periodic	.	.
		N	Level C - Continuous	.	.
16. Verify properties of thin-bed mortar for AAC masonry (first 5000 SF of AAC masonry)	Field inspection	N	Continuous	.	.
17. Verify properties of thin-bed mortar for AAC masonry (after the first 5000 SF of AAC masonry)	Field inspection	N	Level B - Periodic	.	.
		N	Level C - Continuous	.	.
18. Prepare grout and mortar specimens	Field testing	Y	Level B - Periodic	1	.
		N	Level C - Continuous	.	.
19. Observe preparation of prisms	Field inspection	Y	Level B - Periodic	1	.
		N	Level C - Continuous	.	.
1705.5 Wood Construction					
1. Inspection of the fabrication process of wood structural elements and assemblies in accordance with Section 1704.2.5	In-plat review (3)	N	Periodic	.	.
2. For high-load diaphragms, verify grade and thickness of structural panel sheathing agree with approved building plans	Field inspection	N	Periodic	.	.

SCHEDULE OF SPECIAL INSPECTION SERVICES

PROJECT

MATERIAL / ACTIVITY	SERVICE	APPLICABLE TO THIS PROJECT Y/N	EXTENT	AGENT*	DATE COMPLETED
3. For high-load diaphragms, verify nominal size of framing members at adjoining panel edges, nail or staple diameter and length, number of fastener lines, and that spacing between fasteners in each line and at edge margins agree with approved building plans	Field inspection	N	Periodic	.	.
4. Metal-plate-connected wood trusses spanning 60 feet or greater: verify temporary and permanent restraint/bracing are installed in accordance with the approved truss submittal package	Field inspection	N	Periodic	.	.
1705.6 Soils					
1. Verify materials below shallow foundations are adequate to achieve the design bearing capacity.	Field inspection	Y	Periodic	1	.
2. Verify excavations are extended to proper depth and have reached proper material.	Field inspection	Y	Periodic	1	.
3. Perform classification and testing of controlled fit materials.	Field inspection	Y	Periodic	1	.
4. Verify use of proper materials, densities, and lift thicknesses during placement and compaction of controlled fill	Field inspection	Y	Continuous	1	.
5. Prior to placement of controlled fill, observe subgrade and verify that site has been prepared properly	Field inspection	Y	Periodic	1	.
1705.7 Driven Deep Foundations					
1. Verify element materials, sizes and lengths comply with requirements	Field inspection	N	Continuous	.	.
2. Determine capacities of test elements and conduct additional load test, as required	Field inspection	N	Continuous		
3. Observe driving operations and maintain complete and accurate records for each element	Field inspection	N	Continuous		
4. Verify placement locations and plumbness, confirm type and size of hammer ,record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any damage to foundation element	Field inspection	N	Continuous	.	.
5. For steel elements, perform additional inspections per Section 1705.2	See Section 1705.2	N	See Section 1705.2	.	.
6. For concrete elements and concrete-filled elements, perform additional inspections per Section 1705.3	See Section 1705.3	N	See Section 1705.3	.	.
7. For specialty elements, perform additional inspections as determined by the registered design professional in responsible charge	Field inspection	N	In Accordance with Construction Documents	.	.
8. Perform additional inspections and tests in accordance with the construction documents	Field inspection and testing	N	In Accordance with Construction Documents	.	.
1705.8 Cast-in-Place Deep Foundations					
1. Observe drilling operations and maintain complete and accurate records for each element	Field inspection	N	Continuous	.	.
2. Verify placement locations and plumbness, confirm element diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end-bearing strata capacity. Record concrete or grout volumes	Field inspection	N	Continuous	.	.
3. For concrete elements, perform additional inspections in accordance with Section 1705.3	See Section 1705.3	N	See Section 1705.3	.	.
4. Perform additional inspections and tests in accordance with the construction documents	Field inspection and testing	N	In Accordance with Construction Documents	.	.
1705.9 Helical Pile Foundations					
1. Verify installation equipment, pile dimensions, tip elevations, final depth, final installation torque and other data is required	Field inspection	N	Continuous	-	.
2. Perform additional inspections and tests in accordance with the construction documents	Field inspection and testing	N	In Accordance with Construction Documents	-	.
1705.10.1 Structural Wood Special Inspections For Wind Resistance					
1. Inspection of field gluing operations of elements of the main windforce-resisting system	Field inspection	N	Continuous	.	.
2. Inspection of nailing, bolting, anchoring and other fastening of components within the main windforce-resisting system	Shop (3) and field inspection	N	Periodic	.	.
1705.10.2 Cold-formed Steel Special Inspections For Wind Resistance					
1. Inspection during welding operations of elements of the main windforce-resisting system	Shop (3) and field inspection	N	Periodic	.	.
2. Inspections for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system	Shop (3) and field inspection	N	Periodic	.	.
1705.10.3 Wind-resisting Components					
1. Roof cladding	Shop (3) and field inspection	N	Periodic	.	.
2. Wall cladding	Shop (3) and field inspection	N	Periodic	.	.
1705.11.1 Structural Steel Special Inspections for Seismic Resistance					
Inspection of structural steel in accordance with AISC 341	Shop (3) and field inspection	N	In Accordance with ASCE 341	.	.
1705.11.2 Structural Wood Special Inspections for Seismic Resistance					
1. Inspection of field gluing operations of elements of the seismic-force resisting system	Field inspection	N	Continuous	.	.
2. Inspection of nailing, bolting, anchoring and other fastening of components within the seismic-force-resisting system	Shop (3) and field inspection	N	Periodic	.	.
1705.11.3 Cold-formed Steel Light-Frame Construction Special Inspections for Seismic Resistance					
1. Inspection during welding operations of elements of the seismic-force-resisting system	Shop (3) and field inspection	N	Periodic	.	.
2. Inspections for screw attachment, bolting, anchoring and other fastening of components within the seismic-force-resisting system	Shop (3) and field inspection	N	Periodic	.	.
1705.11.4 Designated Seismic Systems Verification					
Inspect and verify that the component label, anchorage or mounting conforms to the certificate of compliance in accordance with Section 1705.12.3	Field inspection	N	Periodic	.	.

SCHEDULE OF SPECIAL INSPECTION SERVICES

PROJECT

MATERIAL / ACTIVITY	SERVICE	APPLICABLE TO THIS PROJECT Y/N	EXTENT	AGENT*	DATE COMPLETED
1705.11.5 Architectural Components Special Inspections for Seismic Resistance					
1. Inspection during the erection and fastening of exterior cladding and interior and exterior veneer	Field inspection	N	Periodic	.	.
2. Inspection during the erection and fastening of interior and exterior nonbearing walls	Field inspection	N	Periodic	.	.
3. Inspection during anchorage of access floors	Field inspection	N	Periodic	.	.
1705.11.6 Mechanical and Electrical Components Special Inspections for Seismic Resistance					
1. Inspection during anchorage of electrical equipment for emergency or standby power systems	Field inspection	N	Periodic	1	.
2. Inspection during the anchorage of other electrical equipment	Field inspection	N	Periodic	.	.
3. Inspection during installation and anchorage of piping systems designed to carry hazardous materials, and their associated mechanical units	Field inspection	N	Periodic	1	.
4. Inspection during the installation and anchorage of HVAC ductwork that will contain hazardous materials	Field inspection	N	Periodic	.	.
5. Inspection during the installation and anchorage of vibration isolation systems	Field inspection	N	Periodic	.	.
1705.11.7 Storage Racks Special Inspection for Seismic Resistance					
Inspection during the anchorage of storage racks 8 feet or greater in height	Field inspection	N	Periodic	.	.
1705.11.8 Seismic Isolation Systems					
Inspection during the fabrication and installation of isolator units and energy dissipation devices used as part of the seismic isolation system	Shop and field inspection	N	Periodic	.	.
1705.12.1 Concrete Reinforcement Testing and Qualification for Seismic Resistance					
1. Review certified mill test reports for each shipment of reinforcement used to resist earthquake-induced flexural and axial forces in reinforced concrete special moment frames, special structural walls, and coupling beams connecting special structural walls	Review certified mill test reports	N	Each Shipment	.	.
2. Verify reinforcement weldability of ASTM A615 reinforcement used to resist earthquake-induced flexural and axial forces in reinforced concrete special moments frames, special structural walls, and coupling beams connecting special structural walls	Review test reports	N	Each Shipment	.	.
1705.12.2 Structural Steel Testing and Qualification for Seismic Resistance					
Test in accordance with the quality assurance requirements of AISC 341	Shop (3) and field testing	N	Per AISC 341	.	.
1705.12.3 Seismic Certifcation of Nonstructural Components					
Review certificate of compliance for designed seismic system components	Certificate of compliance review	N	Each Submittal	.	.
1705.12.4 Seismic Isolation Systems					
Test seismic isolation systems in accordance with ASCE 7, Section 17.8	Prototype testing	N	Per ASCE 7	.	.
1705.13 Sprayed Fire-resistant Materials					
1. Verify surface condition preparation of structural members	Field inspection	N	Periodic	.	.
2. Verify application of sprayed fire-resistant materials	Field inspection	N	Periodic	.	.
3.Verify average thickness of sprayed fire-resistant materials applied to structural members	Field inspection	N	Periodic	.	.
4. Verify density of sprayed fire-resistant material complies with approved fire-resistant design	Field inspection and testing	N	Per IBC Section 1705.13.5	.	.
5. Verify the cohesive/adhesive bond strength of the cured sprayed fire-resistant material	Field inspection and testing	N	Per IBC Section 1705.13.6	.	.
1705.14 Mastic and Intumescent Fire-resistant Coatings					
Inspect mastic and intumescent fire-resistant coatings applied to structural elements and decks	Field inspection	N	Periodic	.	.
1705.15 Exterior Insulation and Finish Systems (EIFS)					
1.Verify materials, details and installations are per the approved construction documents	Field inspection	N	Periodic	.	.
2. Inspection of water-resistive barrier over sheathing substrate	Field inspection	N	Periodic	.	.
1705.16 Fire-resistant Penetrations and Joints					
1. Inspect penetration firestop	Field testing	N	Per ASTM E2174	.	.
2. Inspect fire-resistant joint systems	Field testing	N	Per ASTM E2393	.	.
1705.17 Smoke Control Systems					
1. Leakage testing and recording of device locations prior to concealment	Field testing	N	Periodic	.	.
2. Prior to occupancy and after sufficient completion, pressure difference testing, flow measurements, and detection and control vertification	Field testing	N	Periodic	.	.

* INSPECTION AGENTS	FIRM	ADDRESS	TELEPHONE NO.
1. To be determined			
2.			
3.			
4.			

Notes:
1. *The inspection and testing agent(s) shall be engaged by the Owner or the Owner's Agent, and not by the Contractor orsubcontractor whose work is to be inspected or tested. Any conflict of interest must be disclosed to the Building Official prior to commencing work. The qualifications of the Special Inspector(s) and/or testing agencies may be subject to the approval of the Building Official and/or the Design Professional.*
2. *The list of Special Inspectors may be submitted as a separate document, if noted so above.*
3. *Special Inspections as required by Section 1704.2.5 are not required where the fabricator is approved in accordance with IBC Section 1704.2.5.2*
4. *Observe on a random basis, operations need not be delayed pending these inspections. Perform these tasks on each welded joint, bolted connection, or steel element.*
5. *NDT of welds competed in an approved fabricator's shop may be performed by that fabricator when approved by the AHJ. Refer to AISC 360, N7*

Encircle "Yes" or "No" as appropriate and date this document below:
Are Requirements for Seismic Resistance included in the Statement of Special Inspections? Yes (No)
Are Requirements for Wind Resistance included in the Statement of Special Inspections? Yes (No)

DATE: 2024-06-28

EVIEWING DRAWINGS FOR REFERENCE ONLY

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Revisions

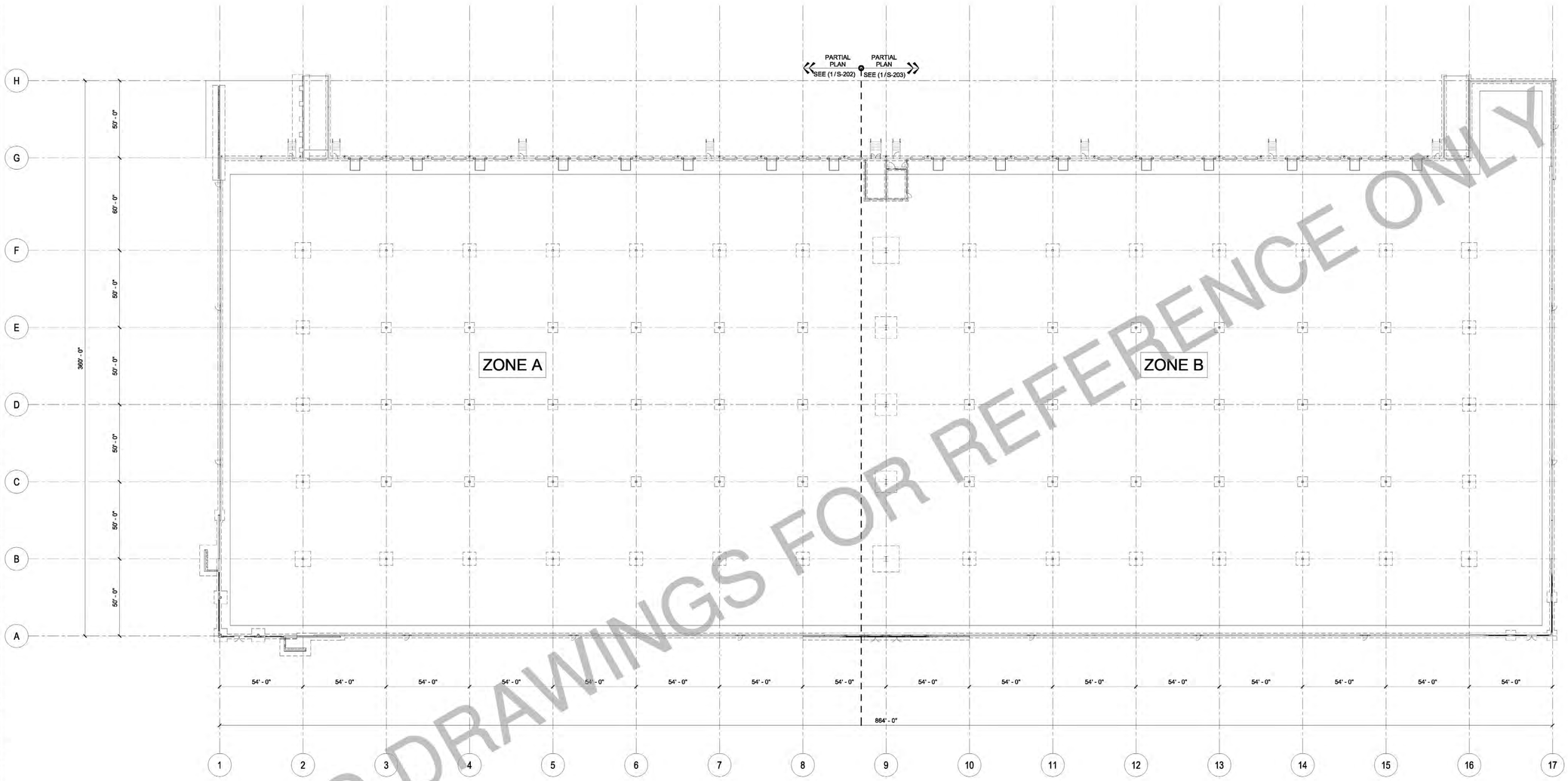
1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
SPECIAL INSPECTIONS

Sheet No.
S-104

CONSTRUCTION SET



EXISTING DRAWINGS FOR REFERENCE ONLY

1 S-201 **OVERALL FOUNDATION PLAN**

SCALE: 1" = 30'-0"

FOUNDATION PLAN NOTES:

1. TOP OF SLAB ON GRADE ELEVATION = SEE CIVIL , REFERENCE LEVEL (+0'-0")
2. CONTROL JOINTS WILL BE SPACED AT EACH COLUMN AND 15 FEET OC MAXIMUM.
3. TOP OF COLUMN FOOTING ELEVATION = (-) 1 FOOT BELOW TOP OF SLAB (UNLESS NOTED OTHERWISE). PLACE SLAB ON GRADE OVER VAPOR RETARDER AND PREPARED SUBGRADE PER GEOTECHNICAL REPORT AND ARCH DRAWINGS.
4. SLAB ON GRADE TO BE 7 INCH UNREINFORCED SLAB BEARING ON STRUCTURAL FILL, ON 10 MIL VAPOR RETARDER, ON COMPACTED SUBGRADE, PER GEOTECHNICAL REPORT, WITH DOWEL BASKETS IN SPEED BAYS ONLY. SLAB TO RECEIVE HARD TROWEL FINISH AND LASER SCREED SHALL BE UTILIZED TO ATTAIN MINIMUM LOCAL & OVERALL SLAB TOLERANCE OF FF50/FL35.

N

FLOOR KEY PLAN

ZONE A S-202	ZONE B S-203

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1	2024 MAY 26	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

Date	Job No.
2024 JUNE 28	24.0000849.000

Sheet Title

OVERALL FOUNDATION PLAN

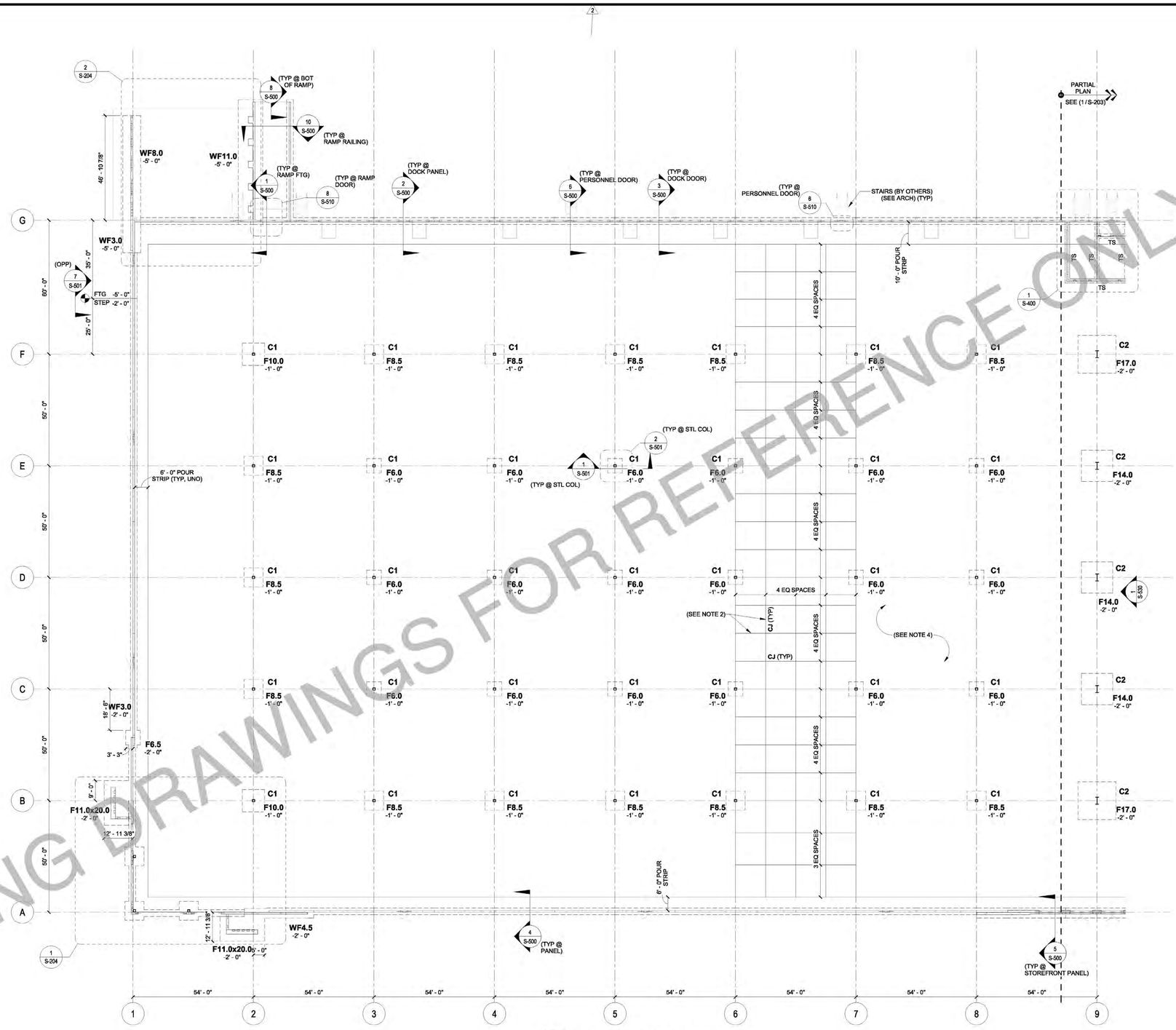
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S-201

CONSTRUCTION SET

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1 S-202 **PARTIAL FOUNDATION PLAN - ZONE A**
SCALE: 1" = 20'-0"

FOUNDATION PLAN NOTES:

1. TOP OF SLAB ON GRADE ELEVATION = SEE CIVIL , REFERENCE LEVEL (+0'-0")
2. CONTROL JOINTS WILL BE SPACED AT EACH COLUMN AND 15 FEET OC MAXIMUM.
3. TOP OF COLUMN FOOTING ELEVATION = (-) 1 FOOT BELOW TOP OF SLAB (UNLESS NOTED OTHERWISE). PLACE SLAB ON GRADE OVER VAPOR RETARDER AND PREPARED SUBGRADE PER GEOTECHNICAL REPORT AND ARCH DRAWINGS.
4. SLAB ON GRADE TO BE 7 INCH UNREINFORCED SLAB BEARING ON STRUCTURAL FILL, ON 10 MIL VAPOR RETARDER, ON COMPACTED SUBGRADE, PER GEOTECHNICAL REPORT, WITH DOWEL BASKETS IN SPEED BAYS ONLY. SLAB TO RECEIVE HARD TROWEL FINISH AND LASER SCREED SHALL BE UTILIZED TO ATTAIN MINIMUM LOCAL & OVERALL SLAB TOLERANCE OF FF50/FL35.

FOUNDATION PLAN LEGEND:

CJ	INDICATES SLAB ON GRADE CONTROL JOINTS. SEE GENERAL NOTES.
TS	INDICATES THICKENED SLAB
TD	INDICATES TURNED DOWN SLAB.
C1	INDICATES COLUMN MARK. SEE COLUMN SCHEDULE.
F5	INDICATES FOOTING MARK. SEE FOOTING SCHEDULE.
BF	INDICATES BRACE FRAME.
-X' - X"	INDICATES TOP OF FOOTING ELEVATION RELATIVE TO FINISH FLOOR ELEVATION.
FTG ###'-##" / STEP ###'-##"	INDICATES STEP IN FOOTING.

FOOTING SCHEDULE

MARK	LENGTH x WIDTH	DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
F6.0	6' - 0" x 6' - 0"	1' - 4"	#6 @12" EW BOTT	2500 PSF	
F6.5	6' - 6" x 6' - 6"	1' - 8"	#6 @12" EW BOTT	2500 PSF	
F8.5	8' - 6" x 8' - 6"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F10.0	10' - 0" x 10' - 0"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F11.0x20.0	11' - 0" x 20' - 0"	1' - 6"	#6 @12" EW TOP; #6 @12" EW BOTT	2500 PSF	
F14.0	14' - 0" x 14' - 0"	2' - 6"	(12) #6 EW T&B;	2500 PSF	
F17.0	17' - 0" x 17' - 0"	2' - 6"	(16) #6 EW T&B;	2500 PSF	

WALL FOUNDATION SCHEDULE

MARK	WIDTH x DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
WF3.0	3' - 0" x 1' - 6"	(3) #5 TOP & BOTT CONT, #3 TIES @ 24"	2500 PSF	
WF4.5	4' - 6" x 1' - 6"	(5) #5 TOP & BOTT CONT, #3 TIES @ 18"	2500 PSF	
WF8.0	8' - 0" x 2' - 0"	(6) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	
WF11.0	11' - 0" x 2' - 0"	(11) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	

COLUMN SCHEDULE

MARK	SIZE	BASE PLATE
C1	HSS12X12X1/4	BP1
C2	W36X247	BP2
C3	HSS12X12X5/16	BP1
C4	W14X53	BP3
C5	HSS12X12X1/2	BP1

N

FLOOR KEY PLAN

ZONE A	ZONE B
S-202	S-203

EXISTING DRAWINGS FOR REFERENCE ONLY

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1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

Date 2024 JUNE 28
Job No. 24.0000849.000

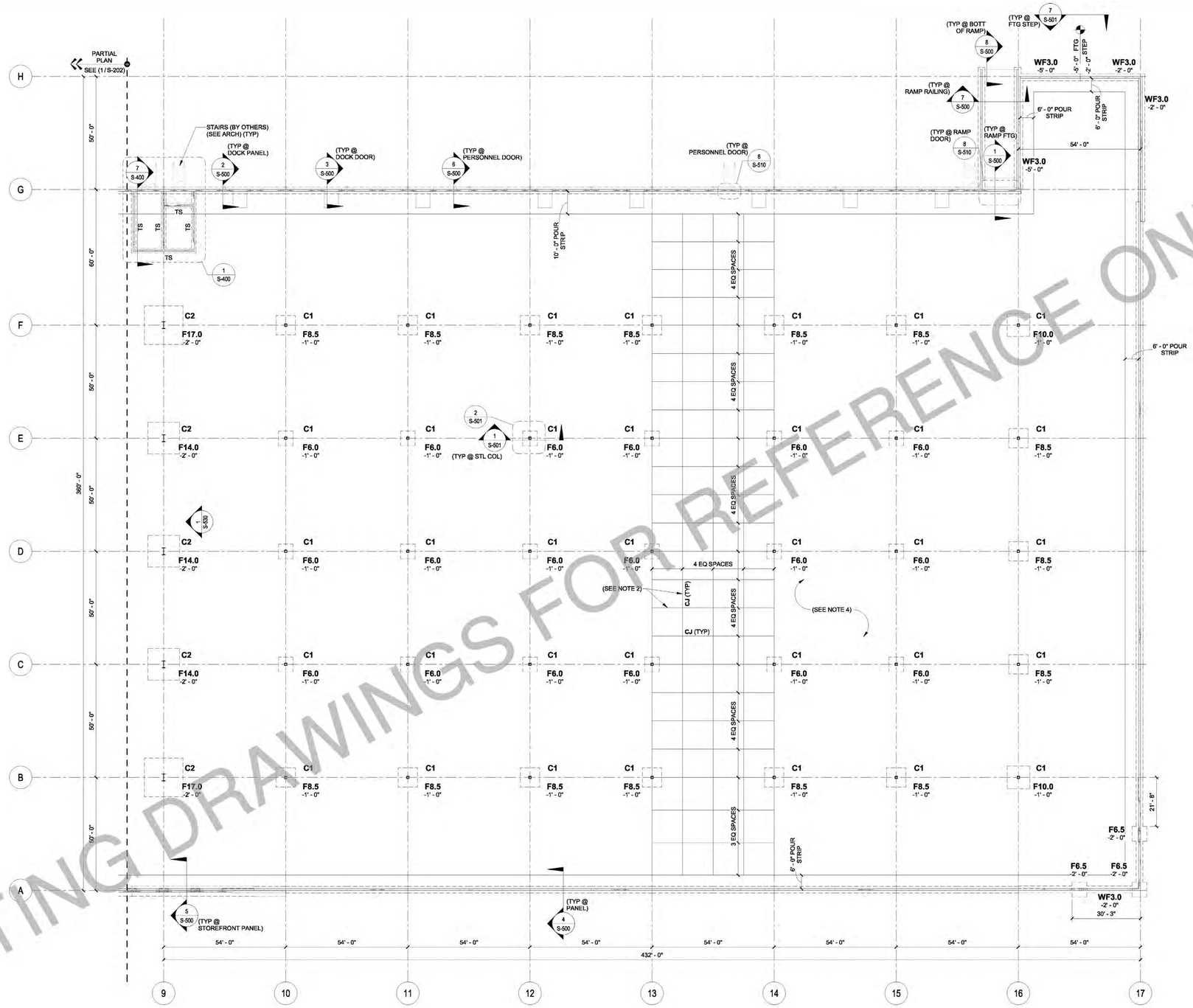
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PARTIAL FOUNDATION PLAN - ZONE A

Sheet No.
S-202

CONSTRUCTION SET

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EXISTING DRAWINGS FOR REFERENCE ONLY

1 S-203 PARTIAL FOUNDATION PLAN - ZONE B

SCALE: 1" = 20'-0"

FOUNDATION PLAN NOTES:

1. TOP OF SLAB ON GRADE ELEVATION = SEE CIVIL , REFERENCE LEVEL (+0'-0")
2. CONTROL JOINTS WILL BE SPACED AT EACH COLUMN AND 15 FEET OC MAXIMUM.
3. TOP OF COLUMN FOOTING ELEVATION = (-) 1 FOOT BELOW TOP OF SLAB (UNLESS NOTED OTHERWISE). PLACE SLAB ON GRADE OVER VAPOR RETARDER AND PREPARED SUBGRADE PER GEOTECHNICAL REPORT AND ARCH DRAWINGS.
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FOUNDATION PLAN LEGEND:

CJ	INDICATES SLAB ON GRADE CONTROL JOINTS. SEE GENERAL NOTES.
TS	INDICATES THICKENED SLAB
TD	INDICATES TURNED DOWN SLAB.
C1	INDICATES COLUMN MARK. SEE COLUMN SCHEDULE.
F5	INDICATES FOOTING MARK. SEE FOOTING SCHEDULE.
BF	INDICATES BRACE FRAME.
-X' - X"	INDICATES TOP OF FOOTING ELEVATION RELATIVE TO FINISH FLOOR ELEVATION.
FTG ###'-##" / STEP ###'-##"	INDICATES STEP IN FOOTING.

FOOTING SCHEDULE

MARK	LENGTH x WIDTH	DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
F6.0	6' - 6" x 6' - 6"	1' - 4"	#6 @12" EW BOTT	2500 PSF	
F8.5	8' - 6" x 8' - 6"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F8.5	8' - 6" x 8' - 6"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F10.0	10' - 0" x 10' - 0"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F11.0x20.0	11' - 0" x 20' - 0"	1' - 6"	#6 @12" EW TOP; #6 @12" EW BOTT	2500 PSF	
F14.0	14' - 0" x 14' - 0"	2' - 6"	(12) #6 EW T&B;	2500 PSF	
F17.0	17' - 0" x 17' - 0"	2' - 6"	(16) #6 EW T&B;	2500 PSF	

WALL FOUNDATION SCHEDULE

MARK	WIDTH x DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
WF3.0	3' - 0" x 1' - 6"	(3) #5 TOP & BOTT CONT, #3 TIES @ 24"	2500 PSF	
WF4.5	4' - 6" x 1' - 6"	(5) #5 TOP & BOTT CONT, #3 TIES @ 18"	2500 PSF	
WF8.0	8' - 0" x 2' - 0"	(8) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	
WF11.0	11' - 0" x 2' - 0"	(11) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	

COLUMN SCHEDULE

MARK	SIZE	BASE PLATE
C1	HSS12X12X1/4	BP1
C2	W36X247	BP2
C3	HSS12X12X5/16	BP1
C4	W14X53	BP3
C5	HSS12X12X1/2	BP1

N

FLOOR KEY PLAN

ZONE A S-202	ZONE B S-203

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2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

Date	Job No.
2024 JUNE 28	24.0000849.000

Sheet Title

PARTIAL FOUNDATION PLAN - ZONE B

Sheet No.

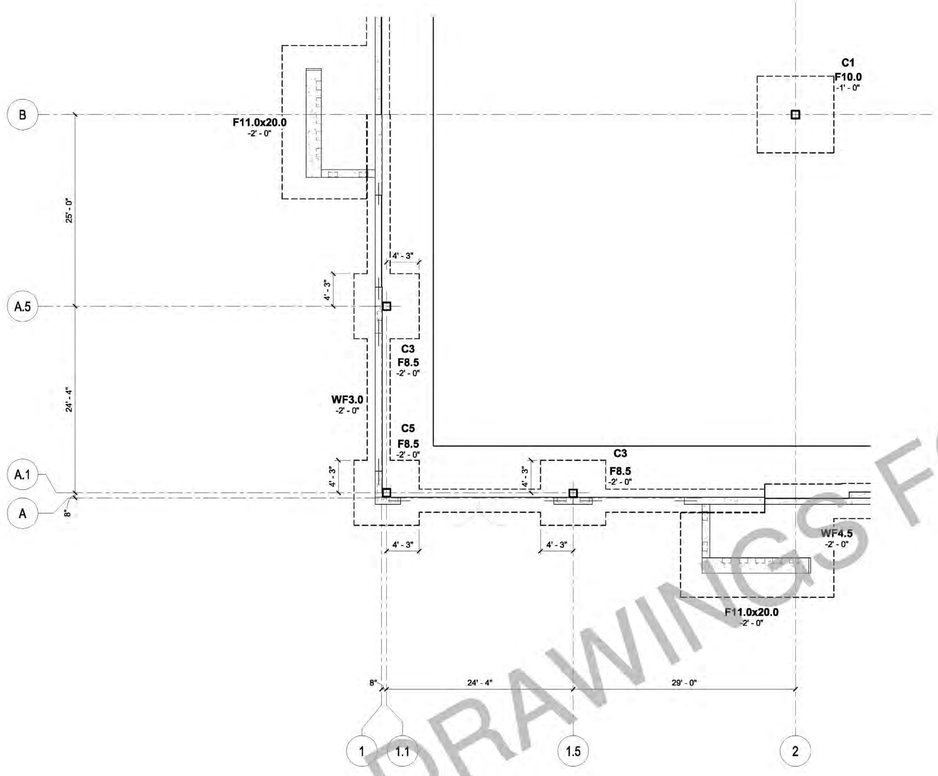
S-203

CONSTRUCTION SET

9.113

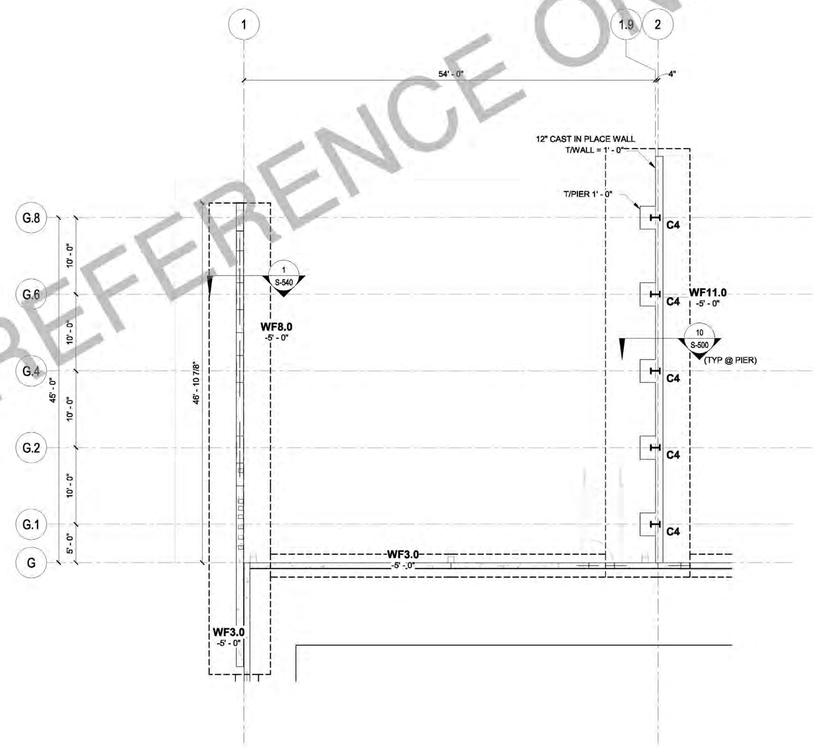
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EXISTING DRAWINGS FOR REFERENCE ONLY



1 S-204 **PARTIAL FOUNDATION PLAN - NORTHWEST CORNER ENTRANCE**

SCALE: 1/8" = 1'-0"



2 S-204 **PARTIAL FOUNDATION PLAN - PICKLEBALL COURT**

SCALE: 1/8" = 1'-0"

FOUNDATION PLAN NOTES:

1. TOP OF SLAB ON GRADE ELEVATION = SEE CIVIL , REFERENCE LEVEL (+0'-0")
2. CONTROL JOINTS WILL BE SPACED AT EACH COLUMN AND 15 FEET OC MAXIMUM.
3. TOP OF COLUMN FOOTING ELEVATION = (-) 1 FOOT BELOW TOP OF SLAB (UNLESS NOTED OTHERWISE). PLACE SLAB ON GRADE OVER VAPOR RETARDER AND PREPARED SUBGRADE PER GEOTECHNICAL REPORT AND ARCH DRAWINGS.
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FOUNDATION PLAN LEGEND:

CJ	INDICATES SLAB ON GRADE CONTROL JOINTS. SEE GENERAL NOTES.
TS	INDICATES THICKENED SLAB
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F5	INDICATES FOOTING MARK. SEE FOOTING SCHEDULE.
BF	INDICATES BRACE FRAME.
-X' - X"	INDICATES TOP OF FOOTING ELEVATION RELATIVE TO FINISH FLOOR ELEVATION.
FTG ####'-##" / STEP ####'-##"	INDICATES STEP IN FOOTING.

FOOTING SCHEDULE

MARK	LENGTH x WIDTH	DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
F6.0	6' - 6" x 6' - 6"	1' - 4"	#6 @12" EW BOTT	2500 PSF	
F6.5	6' - 6" x 6' - 6"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F8.5	8' - 6" x 8' - 6"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F10.0	10' - 0" x 10' - 0"	1' - 6"	#6 @12" EW BOTT	2500 PSF	
F11.0x20.0	11' - 0" x 20' - 0"	1' - 6"	#6 @12" EW TOP; #6 @12" EW BOTT	2500 PSF	
F14.0	14' - 0" x 14' - 0"	2' - 6"	(12) #6 EW T&B;	2500 PSF	
F17.0	17' - 0" x 17' - 0"	2' - 6"	(16) #6 EW T&B;	2500 PSF	

WALL FOUNDATION SCHEDULE

MARK	WIDTH x DEPTH	REINFORCEMENT	SOIL BEARING	REMARKS
WF3.0	3' - 0" x 1' - 6"	(3) #5 TOP & BOTT CONT, #3 TIES @ 24"	2500 PSF	
WF4.5	4' - 6" x 1' - 6"	(5) #5 TOP & BOTT CONT, #3 TIES @ 18"	2500 PSF	
WF8.0	8' - 0" x 2' - 0"	(8) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	
WF11.0	11' - 0" x 2' - 0"	(11) #7 TOP & BOTT CONT, #3 TIES @ 12" OC	2500 PSF	

COLUMN SCHEDULE

MARK	SIZE	BASE PLATE
C1	HSS12X12X1/4	BP1
C2	W36X247	BP2
C3	HSS12X12X5/16	BP1
C4	W14X53	BP3
C5	HSS12X12X1/2	BP1

N

FLOOR KEY PLAN

ZONE A	ZONE B
S-202	S-203

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2	2024 JUNE 28	CONSTRUCTION SET
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Date 2024 JUNE 28

Job No. 24.000049.000

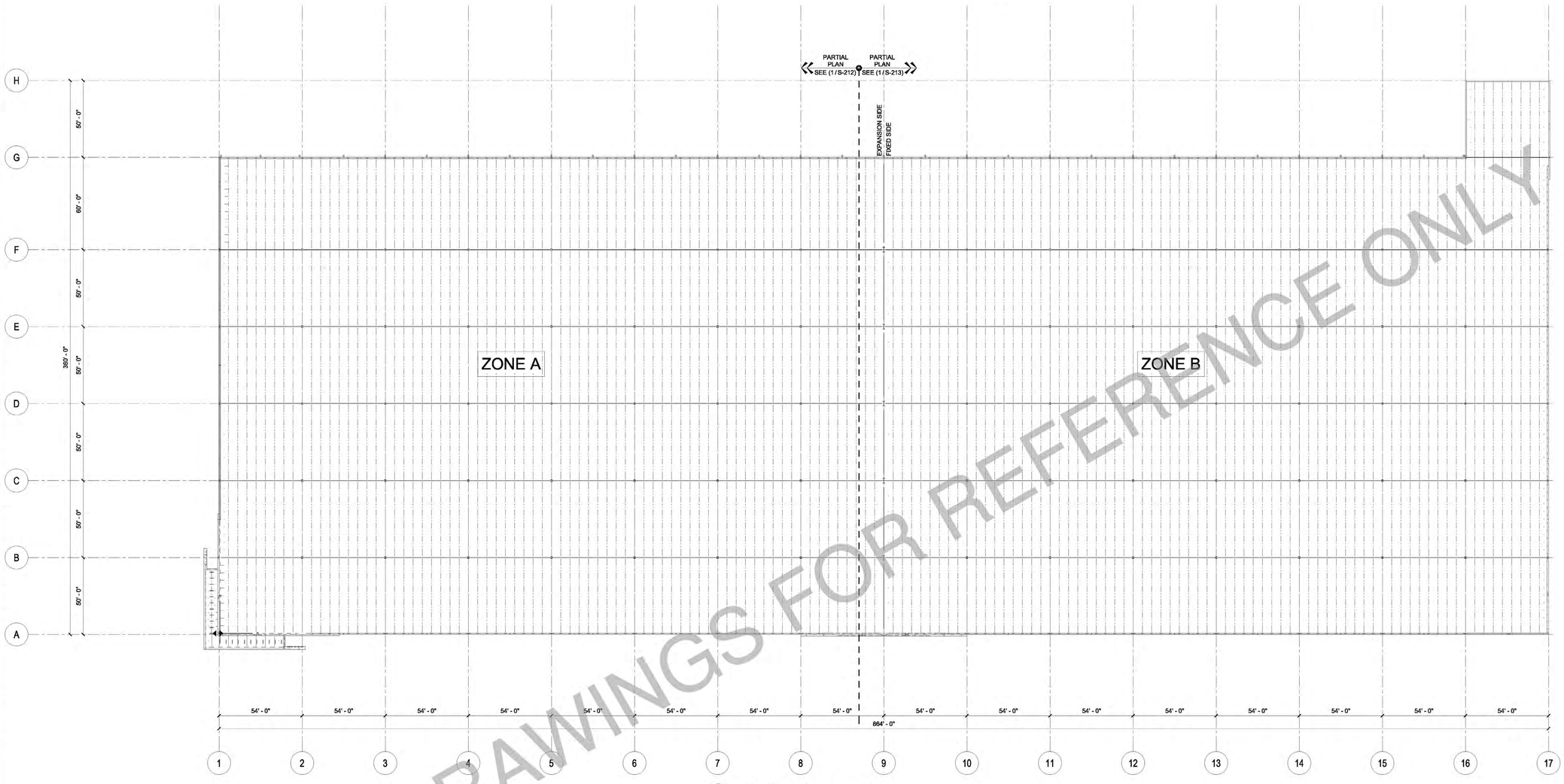
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Sheet No.
S-204 2

CONSTRUCTION SET

9.114

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SHEET NAME: PARTIAL FOUNDATION PLAN - ENLARGED PLANS SHEET NUMBER: S-204 PROJECT NAME: PACE ROAD DISTRIBUTION CENTER - BUILDING 4
FILENAME: C:\Users\BillenO\Bennett&Pless\Documents\24.03.058-Nelson-Pace Road DC-BLDG 4-R23_bgharagozlou\FRTGU.rvt



1 S-211 **OVERALL ROOF FRAMING PLAN**
SCALE: 1" = 30'-0"

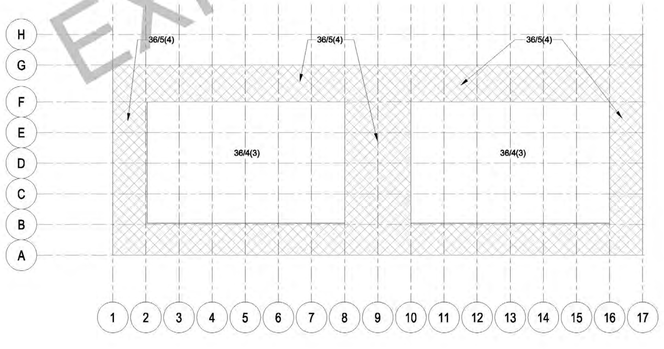
EXISTING DRAWINGS FOR REFERENCE ONLY

ROOF PLAN NOTES:

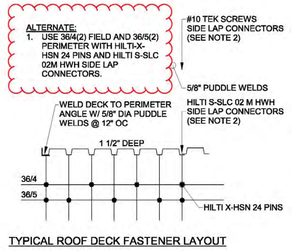
1. ROOF METAL DECK WILL BE 1-1/2 INCH DEEP, 22 GAGE, TYPE B (80 KSI), WIDE RIB PRIMED WHITE (BOTTOM) (MINIMUM 3 SPANS). (SEE ROOF LEGEND)
2. STEEL JOISTS SHALL BE BRACED BY HORIZONTAL AND/OR DIAGONAL BRIDGING AS REQUIRED BY THE STL JST INSTITUTE.
3. STL JST SEATS SHALL BE 1/4 INCH MINIMUM AT WALLS.
4. SEE GENERAL NOTES FOR ADDITIONAL INFORMATION.
5. ELEVATIONS SHOWN ON PLAN ARE BOTTOM OF DECK.
6. SEE TYPICAL ROOF OPENING DETAIL FOR FRAMING OF THE OPENINGS THROUGH THE ROOF. CONTRACTOR TO SUPPLY STEEL FABRICATOR WITH EXACT DIMENSIONS AND LOCATIONS OF OPENINGS THROUGH THE ROOF.
7. JOIST SHOP DRAWINGS SHALL INDICATE WHERE THE BOLTED DIAGONAL BRIDGING IS REQUIRED FOR THE JOISTS. THE JOIST SHOP DRAWINGS SHALL ALSO INDICATE WHERE HOISTING CABLES SHALL NOT BE RELEASED UNTIL THE ROW OF BOLTED DIAGONAL BRIDGING IS COMPLETELY INSTALLED.
8. ROOF STRUCTURE SHOWN AS REQUIRED TO MAINTAIN MIN VERTICAL CLEARANCE OF 36 FEET
9. ROOF SLOPE BASED ON 1/4 INCH PER FOOT.
10. JOISTS AND JOIST GIRDERS SHALL BE CAMBERED PER SJI SPECIFICATIONS.
11. JST GIRDERSELF WEIGHT IS NOT INCLUDED IN POINT LOADS SHOWN. POINT LOADS SHOWN ARE SERVICE (ASD).
12. JOISTS AND JOIST GIRDERS SHALL BE DESIGNED FOR ADDITIONAL LOADS DUE TO SPRINKLER MAINS. GENERAL CONTRACTOR AND JOIST MANUFACTURER SHALL COORDINATE LOCATIONS OF SPRINKLER MAINS. JOIST LOADING INFORMATION SHOWN DOES NOT INCLUDE FIRE PROJECTION LINES GREATER THAN 4 INCHES DIAMETER (SEE S-102 FOR ADDITIONAL INFORMATION).
13. JOISTS AND JOIST GIRDERS SHALL BE DESIGNED FOR ADDITIONAL LOADS DUE TO ROOF TOP EQUIPMENT. GENERAL CONTRACTOR AND JOIST MANUFACTURER SHALL COORDINATE LOCATIONS AND WEIGHTS OF ROOF TOP EQUIPMENT WITH FINAL MECHANICAL DRAWINGS. LOCATIONS, GEOMETRY AND LOADING SHOWN ON PLANS ARE APPROXIMATE. (SEE ROOF LEGEND)
14. FOR BIDDING PURPOSES ASSUME 10% OF JOISTS WILL REQUIRE ADDITIONAL 30 PLF CAPACITY FOR SPRINKLER MAINS AND JOIST GIRDERS WILL REQUIRE ADDITIONAL 1 KIP JOIST REACTION.
15. GENERAL CONTRACTOR TO COORDINATE FIXED EXPANSION SIDE WITH SEQUENCE OF CONSTRUCTION.
16. MINIMUM EXPANSION JOINT AND STRUCTURAL SEPARATION AS REQUIRED BY ASCE 7-22, 12.12.3

BOTTOM OF DECK SCHEDULE

GRIDLINE	ELEVATION	JOIST SEAT DEPTH	REMARKS
B	42' - 10 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
C	41' - 10"	2 1/2"	(TYP-UNO-SEE PLAN)
D	40' - 9 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
E	39' - 9"	2 1/2"	(TYP-UNO-SEE PLAN)
F	38' - 8 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
G	37' - 5 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)

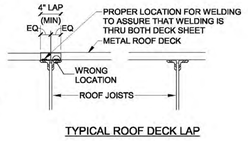


2 S-211 **OVERALL DECK CONNECTION PLAN**
SCALE: 1" = 100'-0"

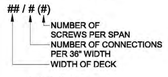


TYPICAL ROOF DECK FASTENER LAYOUT

NOTES:
1. CONNECTION OF ROOF DECK TO JOISTS AND BEAMS SHALL SATISFY THE ULTIMATE LOAD FOR THE ROOFING SYSTEM IN THE CORNER, PERIMETER AND INTERIOR AREAS (SEE ROOFING SPECIFICATIONS).
2. DECK SIDE LAPS SHALL BE NESTABLE.

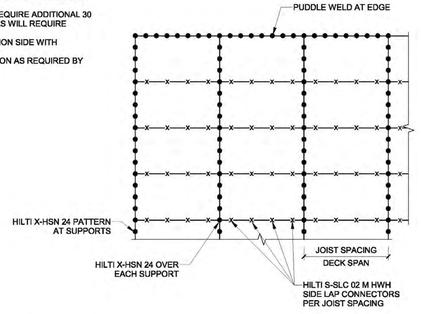


TYPICAL ROOF DECK LAP



LEGEND

3 S-211 **DECK CONNECTION PATTERN DETAIL**
SCALE: 3/4" = 1'-0"



4 S-211 **DECK ATTACHMENT DETAIL**
SCALE: 3/4" = 1'-0"



ROOF KEY PLAN

ZONE A S-212	ZONE B S-213

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
OVERALL ROOF FRAMING PLAN

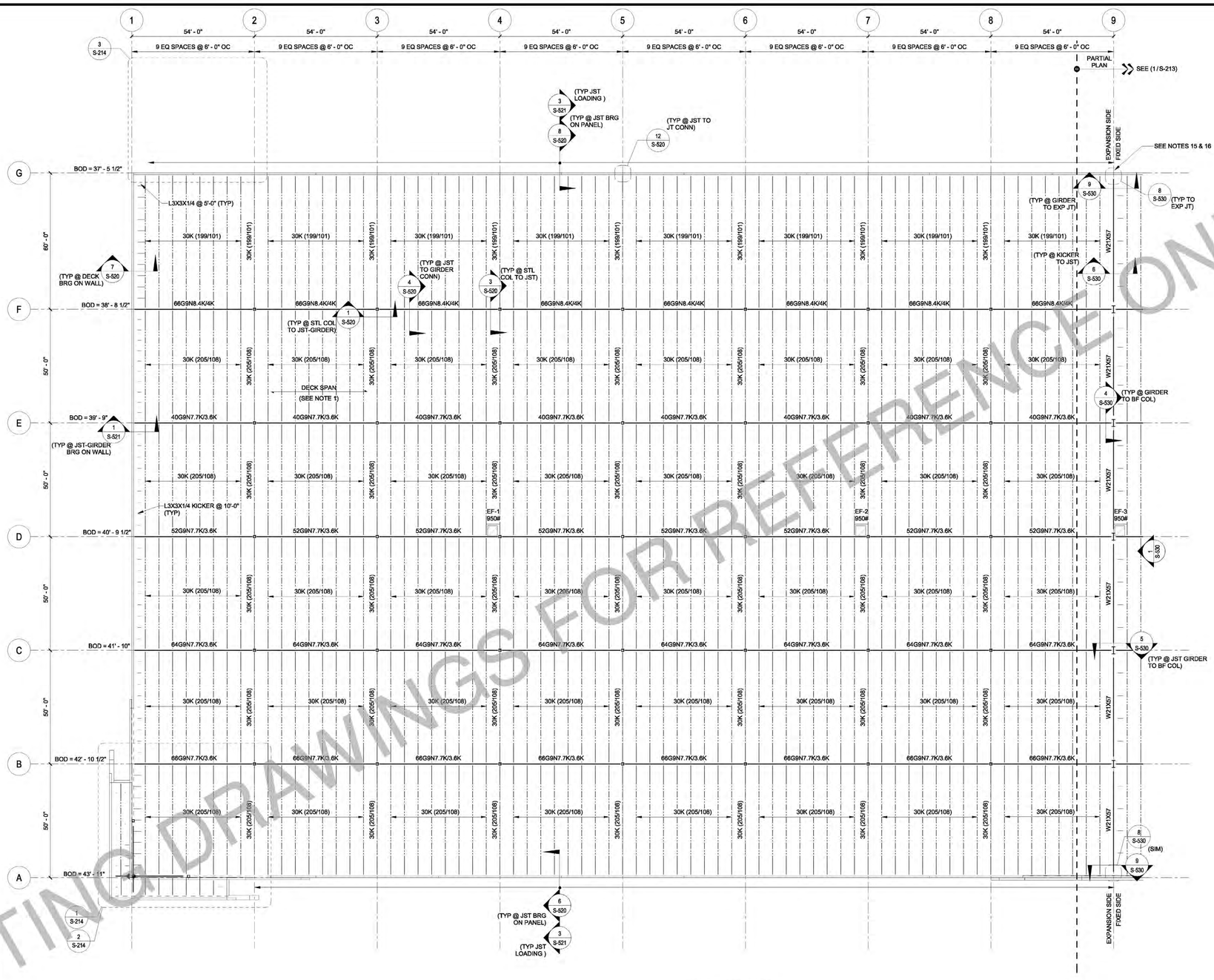
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CONSTRUCTION SET

9.115

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FILENAME: C:\Users\BlakeGhanagodoi\Documents\24.03.058-Nelson-Pace Road DC-BLD0 4-r23_bghanagodoiFRTGU.rvt

EXISTING DRAWINGS FOR REFERENCE ONLY



1 S-212 **PARTIAL ROOF FRMAING PLAN - ZONE A**
SCALE: 1" = 20'-0"

ROOF PLAN NOTES:

1. ROOF METAL DECK WILL BE 1-1/2 INCH DEEP, 22 GAGE, TYPE B (80 KSI), WIDE RIB PRIMED WHITE (BOTTOM) (MINIMUM 3 SPANS). (SEE ROOF LEGEND)
2. STEEL JOISTS SHALL BE BRACED BY HORIZONTAL AND/OR DIAGONAL BRIDGING AS REQUIRED BY THE STL JST INSTITUTE.
3. STL JST SEATS SHALL BE 1/4 INCH MINIMUM AT WALLS.
4. SEE GENERAL NOTES FOR ADDITIONAL INFORMATION.
5. ELEVATIONS SHOWN ON PLAN ARE BOTTOM OF DECK.
6. SEE TYPICAL ROOF OPENING DETAIL FOR FRAMING OF THE OPENINGS THROUGH THE ROOF. CONTRACTOR TO SUPPLY STEEL FABRICATOR WITH EXACT DIMENSIONS AND LOCATIONS OF OPENINGS THROUGH THE ROOF.
7. JOIST SHOP DRAWINGS SHALL INDICATE WHERE THE BOLTED DIAGONAL BRIDGING IS REQUIRED FOR THE JOISTS. THE JOIST SHOP DRAWINGS SHALL ALSO INDICATE WHERE HOISTING CABLES SHALL NOT BE RELEASED UNTIL THE ROW OF BOLTED DIAGONAL BRIDGING IS COMPLETELY INSTALLED.
8. ROOF STRUCTURE SHOWN AS REQUIRED TO MAINTAIN MIN VERTICAL CLEARANCE OF 36 FEET
9. ROOF SLOPE BASED ON 1/4 INCH PER FOOT.
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11. JST GIRDERSELF WEIGHT IS NOT INCLUDED IN POINT LOADS SHOWN. POINT LOADS SHOWN ARE SERVICE (ASD).
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13. JOISTS AND JOIST GIRDERS SHALL BE DESIGNED FOR ADDITIONAL LOADS DUE TO ROOF TOP EQUIPMENT. GENERAL CONTRACTOR AND JOIST MANUFACTURER SHALL COORDINATE LOCATIONS AND WEIGHTS OF ROOF TOP EQUIPMENT WITH FINAL MECHANICAL DRAWINGS. LOCATIONS, GEOMETRY AND LOADING SHOWN ON PLANS ARE APPROXIMATE. (SEE ROOF LEGEND)
14. FOR BIDDING PURPOSES ASSUME 10% OF JOISTS WILL REQUIRE ADDITIONAL 30 PLF CAPACITY FOR SPRINKLER MAINS AND JOIST GIRDERS WILL REQUIRE ADDITIONAL 1 KIP JOIST REACTION.
15. GENERAL CONTRACTOR TO COORDINATE FIXED EXPANSION SIDE WITH SEQUENCE OF CONSTRUCTION.
16. MINIMUM EXPANSION JOINT AND STRUCTURAL SEPARATION AS REQUIRED BY ASCE 7-22, 12.12.3

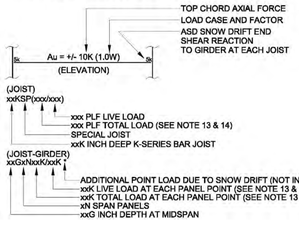
BOTTOM OF DECK SCHEDULE

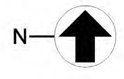
GRIDLINE	ELEVATION	JOIST SEAT DEPTH	REMARKS
B	42' - 10 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
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F	38' - 8 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
G	37' - 5 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)

ROOF PLAN LEGEND

DECK SPAN (SEE ROOF PLAN NOTES)

RTU ZONE (SEE ROOF PLAN NOTES)





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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
6	2024 DECEMBER 06	EXHAUST FAN COORD

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
PARTIAL ROOF FRAMING PLAN - ZONE A

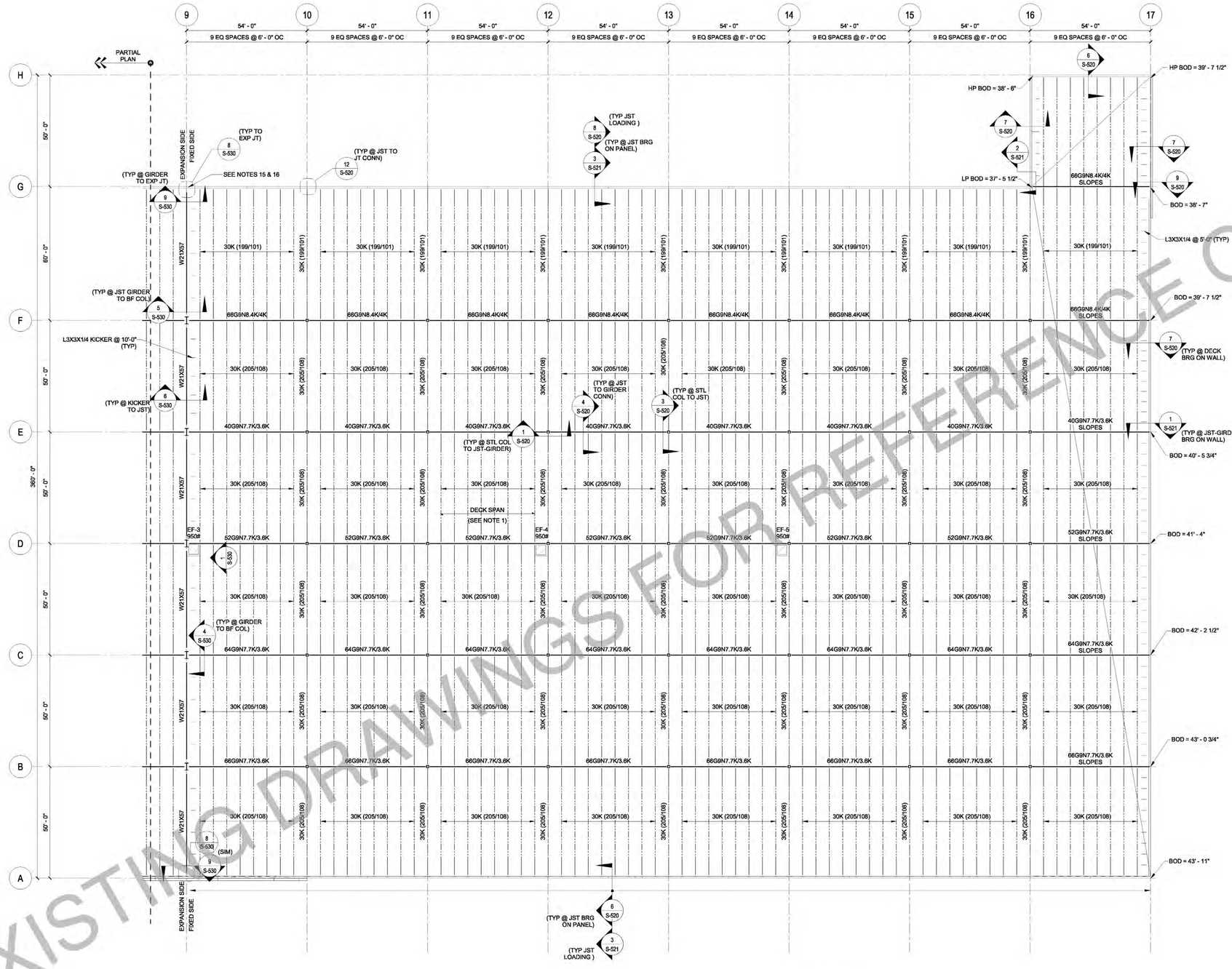
Sheet No.
S-212

CONSTRUCTION SET

9.116

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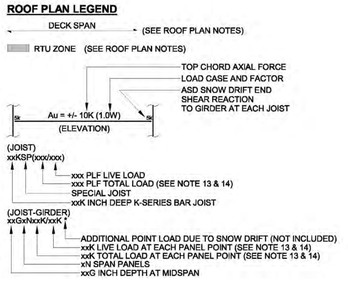
EXISTING DRAWINGS FOR REFERENCE ONLY



1 S-213 **PARTIAL ROOF FRMAING PLAN - ZONE B**

SCALE: 1" = 20'-0"

ROOF PLAN LEGEND



ROOF PLAN NOTES:

1. ROOF METAL DECK WILL BE 1-1/2 INCH DEEP, 22 GAGE, TYPE B (80 KSI), WIDE RIB PRIMED WHITE (BOTTOM) (MINIMUM 3 SPANS). (SEE ROOF LEGEND)
2. STEEL JOISTS SHALL BE BRACED BY HORIZONTAL AND/OR DIAGONAL BRIDGING AS REQUIRED BY THE STL. JST INSTITUTE.
3. STL JST SEATS SHALL BE 1/4 INCH MINIMUM AT WALLS.
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15. GENERAL CONTRACTOR TO COORDINATE FIXED EXPANSION SIDE WITH SEQUENCE OF CONSTRUCTION.
16. MINIMUM EXPANSION JOINT AND STRUCTURAL SEPARATION AS REQUIRED BY ASCE 7-22, 12.12.3

BOTTOM OF DECK SCHEDULE

GRIDLINE	ELEVATION	JOIST SEAT DEPTH	REMARKS
B	42' - 10 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
C	41' - 10"	2 1/2"	(TYP-UNO-SEE PLAN)
D	40' - 9 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
E	39' - 9"	2 1/2"	(TYP-UNO-SEE PLAN)
F	38' - 8 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)
G	37' - 5 1/2"	2 1/2"	(TYP-UNO-SEE PLAN)

N

ROOF KEY PLAN

ZONE A S-212	ZONE B S-213

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
6	2024 DECEMBER 06	EXHAUST FAN COORD

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
PARTIAL ROOF FRAMING PLAN - ZONE B

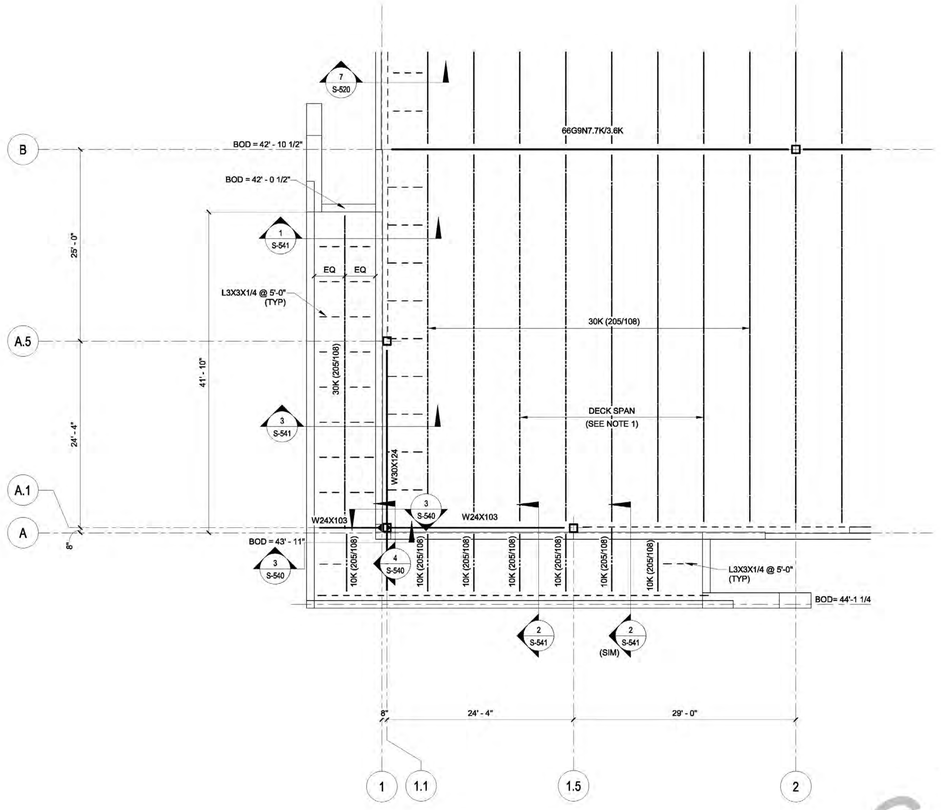
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S-213

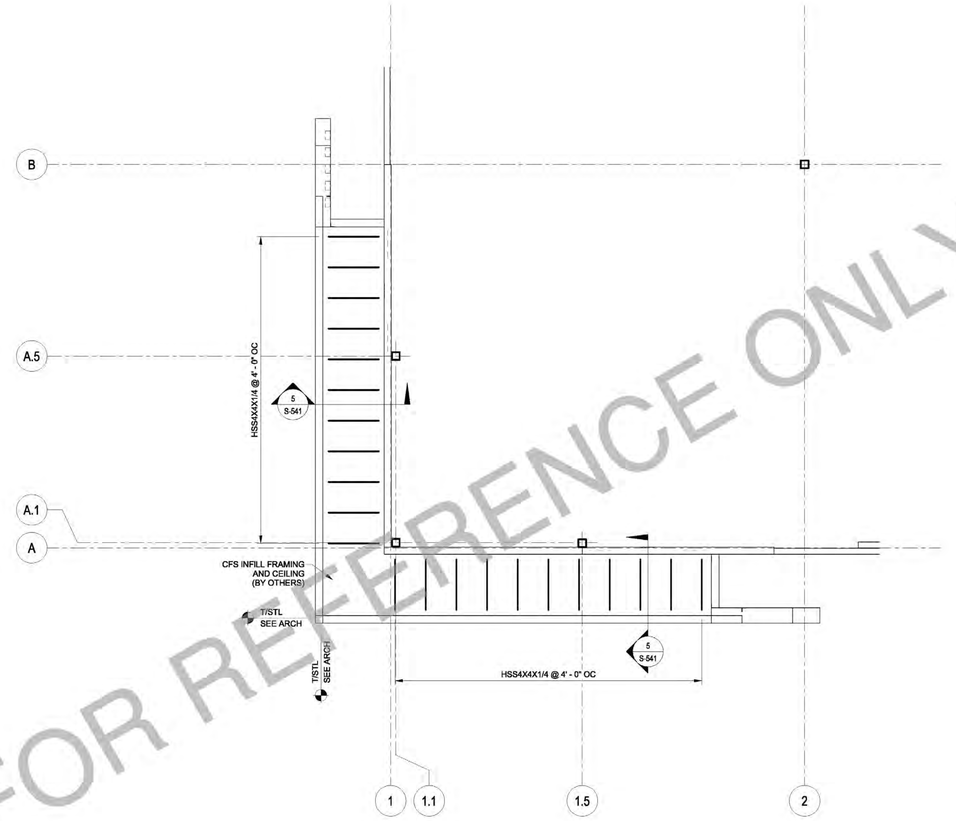
CONSTRUCTION SET

9.117

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EXISTING DRAWINGS FOR REFERENCE ONLY





1 S-214 ROOF FRAMING PLAN - ENLARGED FRONT CORNER

SCALE: 1/8" = 1'-0"

2 S-214 CORNER FRAMING SOFFIT FRAMING PLAN

SCALE: 1/8" = 1'-0"

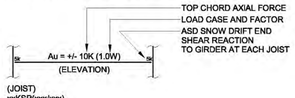
ROOF PLAN NOTES:

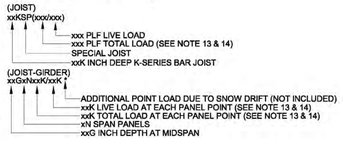
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2. STEEL JOISTS SHALL BE BRACED BY HORIZONTAL AND/OR DIAGONAL BRIDGING AS REQUIRED BY THE STL JST INSTITUTE.
3. STEEL JOIST SEATS SHALL BE 1/4 INCH MINIMUM AT WALLS.
4. SEE GENERAL NOTES FOR ADDITIONAL INFORMATION. SEE ROOF FRAMING NOTES ON S212 FOR NOTES REGARDING JOIST AND JOIST GIRDER LOADING AND FRAMING.
5. ELEVATIONS SHOWN ON PLAN ARE BOTTOM OF DECK.
6. ROOF STRUCTURE SHOWN AS REQUIRED TO MAINTAIN MIN VERTICAL CLEARANCE OF 36 FEET
7. ROOF SLOPE BASED ON 1/4 INCH PER FOOT.
8. MINIMUM EXPANSION JOINT AND STRUCTURAL SEPARATION AS REQUIRED BY ASCE 7-22, 12.12.3

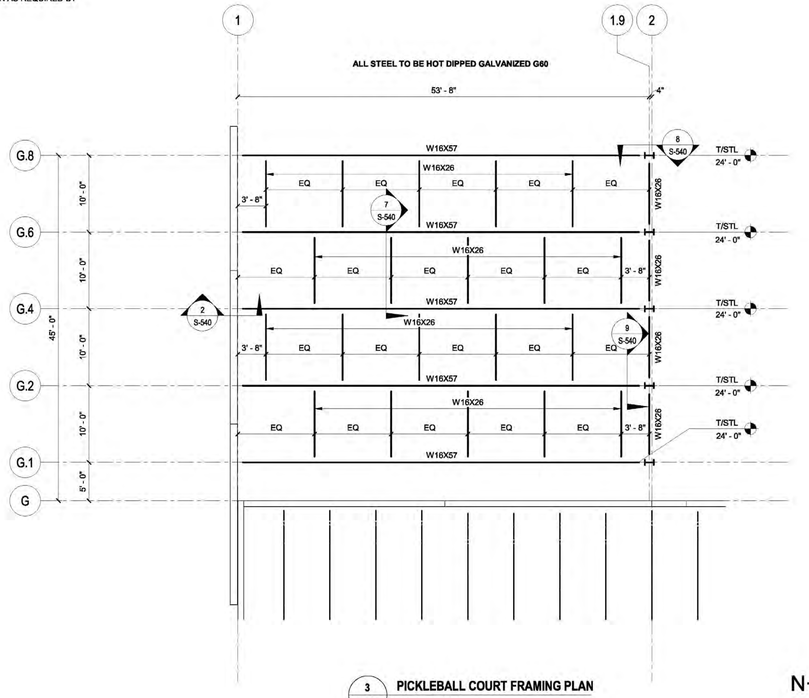
ROOF PLAN LEGEND

DECK SPAN (SEE ROOF PLAN NOTES)

RTU ZONE (SEE ROOF PLAN NOTES)







3 S-214 PICKLEBALL COURT FRAMING PLAN

SCALE: 1/8" = 1'-0"



ROOF KEY PLAN

ZONE A S-212	ZONE B S-213

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2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

Date 2024 JUNE 28
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Sheet Title
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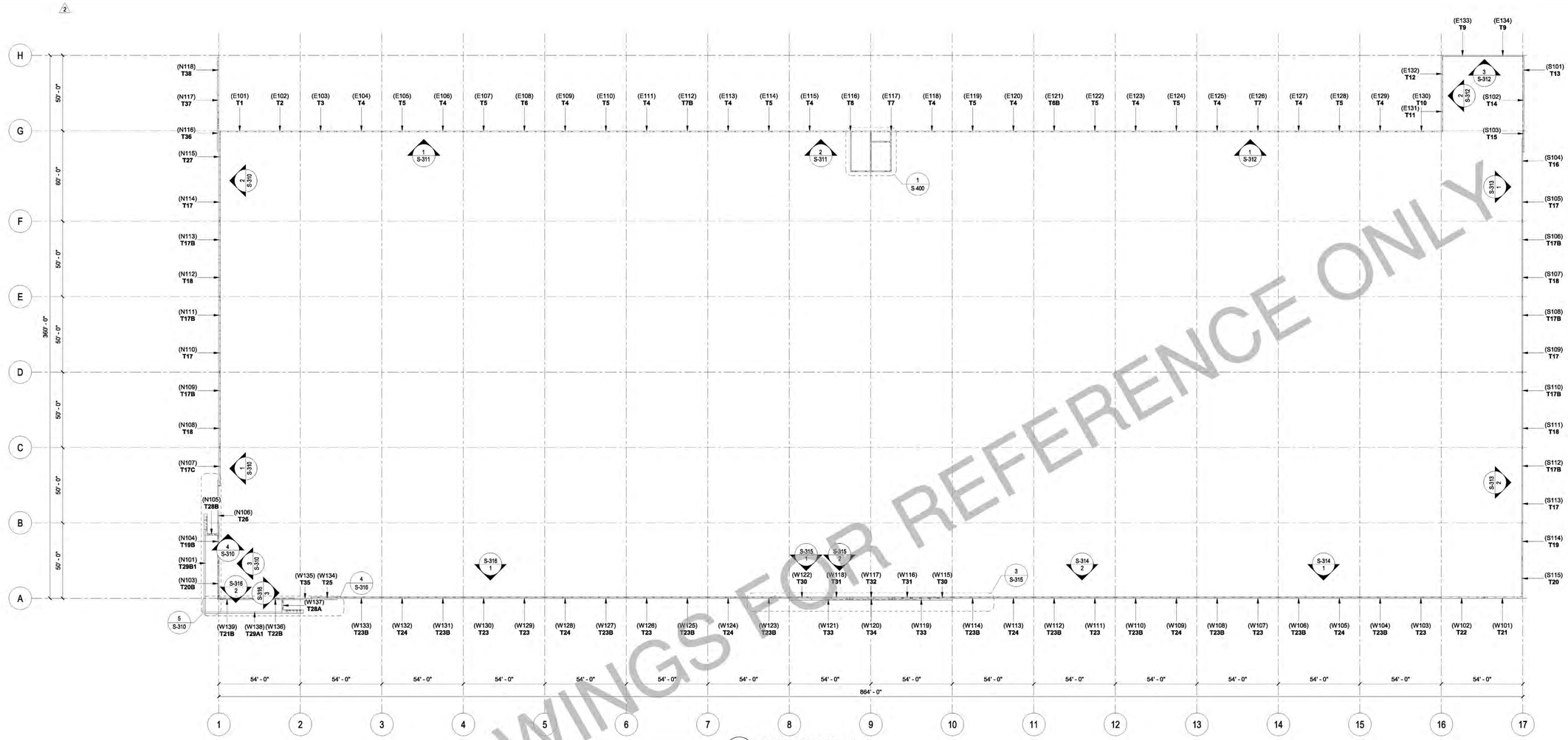
Sheet No.
S-214 2

CONSTRUCTION SET

9.118

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PLOTTED: 2/28/2025 3:27:53 PM

EXISTING DRAWINGS FOR REFERENCE ONLY



TILT PANEL SCHEDULE

T1	TH = 9 1/4"	F'c = 4000 PSI
T2	TH = 9 1/4"	F'c = 4000 PSI
T3	TH = 9 1/4"	F'c = 4000 PSI
T4	TH = 9 1/4"	F'c = 4000 PSI
T5	TH = 9 1/4"	F'c = 4000 PSI
T6	TH = 9 1/4"	F'c = 4000 PSI
T6B	TH = 9 1/4"	F'c = 4000 PSI
T7	TH = 9 1/4"	F'c = 4000 PSI
T7B	TH = 9 1/4"	F'c = 4000 PSI
T8	TH = 9 1/4"	F'c = 4000 PSI
T9	TH = 9 1/4"	F'c = 4000 PSI
T10	TH = 9 1/4"	F'c = 6000 PSI
T11	TH = 9 1/4"	F'c = 4000 PSI
T12	TH = 9 1/4"	F'c = 4000 PSI
T13	TH = 9 1/4"	F'c = 4000 PSI
T14	TH = 9 1/4"	F'c = 4000 PSI
T15	TH = 9 1/4"	F'c = 4000 PSI
T16	TH = 9 1/4"	F'c = 4000 PSI
T17	TH = 9 1/4"	F'c = 4000 PSI
T17B	TH = 9 1/4"	F'c = 4000 PSI
T17C	TH = 9 1/4"	F'c = 4000 PSI
T18	TH = 9 1/4"	F'c = 4000 PSI
T19	TH = 11 1/4"	F'c = 6000 PSI
T19B	TH = 11 1/4"	F'c = 6000 PSI
T20	TH = 11 1/4"	F'c = 6000 PSI
T20B	TH = 11 1/4"	F'c = 6000 PSI
T21	TH = 11 1/4"	F'c = 6000 PSI
T21B	TH = 11 1/4"	F'c = 4000 PSI
T22	TH = 11 1/4"	F'c = 6000 PSI
T22B	TH = 11 1/4"	F'c = 4000 PSI
T23	TH = 9 1/4"	F'c = 4000 PSI
T23B	TH = 9 1/4"	F'c = 4000 PSI
T24	TH = 9 1/4"	F'c = 4000 PSI
T25	TH = 9 1/4"	F'c = 4000 PSI
T26	TH = 9 1/4"	F'c = 4000 PSI
T27	TH = 9 1/4"	F'c = 6000 PSI
T28A	TH = 12"	F'c = 4000 PSI
T28B	TH = 12"	F'c = 4000 PSI
T29A1	TH = 24"	F'c = 4000 PSI
T29A2	TH = 12"	F'c = 4000 PSI
T29B1	TH = 24"	F'c = 4000 PSI
T29B2	TH = 12"	F'c = 4000 PSI
T30	TH = 9 1/4"	F'c = 4000 PSI
T31	TH = 10 1/2"	F'c = 6000 PSI
T32	TH = 10 1/2"	F'c = 6000 PSI
T33	TH = 8"	F'c = 6000 PSI
T34	TH = 8"	F'c = 6000 PSI
T35	TH = 9 1/4"	F'c = 4000 PSI
T36	TH = 11 1/4"	F'c = 4000 PSI
T37	TH = 11 1/4"	F'c = 4000 PSI
T38	TH = 11 1/4"	F'c = 4000 PSI
TPRA	TH = 7 1/4"	F'c = 4000 PSI
TPRB	TH = 7 1/4"	F'c = 4000 PSI
TPRC	TH = 7 1/4"	F'c = 4000 PSI
TPRD	TH = 7 1/4"	F'c = 4000 PSI
TPRE	TH = 7 1/4"	F'c = 4000 PSI

1 S-300 OVERALL TILT PANEL PLAN

SCALE: 1" = 30'-0"

TILT PANEL PLAN NOTES:

1. TILT PANELS ARE DRAWN IN ELEVATION FROM THE INSIDE OF THE BUILDING.
2. TILT PANELS SHOULD REMAIN BRACED UNTIL THE ENTIRETY OF THE ROOF DECK AND ROOF FRAMING HAS BEEN CONNECTED TO THE PANELS, AND THE CONCRETE SLAB POUR STRIP CONCRETE HAS BEEN PLACED. IN LIEU OF POUR STRIP CONCRETE MECHANICAL CONNECTIONS CAN BE MADE FROM THE BASE OF THE TILT PANEL TO THE FOOTING.
3. REFER TO THE ARCHITECTURAL DRAWINGS FOR CONFIRMATION OF ALL PANEL DIMENSIONS, AND ALL REVEALS.
4. SEE TILT PANEL ELEVATIONS FOR PANEL THICKNESSES.
5. SEE TILT PANEL REBAR ELEVATIONS FOR PANEL REINFORCEMENT.
6. ALL REINFORCEMENT TO BE EA FACE UNLESS NOTED OTHERWISE.
7. TYPICAL REINFORCING UNLESS NOTED OTHERWISE ON PANEL REBAR ELEVATIONS:
   A. #5 + HK, EF AT TOP OF PANEL.
   B. (2) #5 AT 8" OC +HK EF @ BOTTOM OF PANEL.
   C. #5 + HK, EF AT TOP OF OPENINGS.
   D. #4 DOWELS AT 18 INCHES OC FOR DOWELS , TYPICAL. PROVIDE MINIMUM OF (3) DOWELS PER LEG.
   E. TIES #3 AT 7 INCHES OC FOR VERTICAL BARS AT PANEL LEGS ADJACENT TO OPENINGS OR FUTURE OPENINGS.
8. SEE DETAILS ON S-520 FOR DETAILS AT GIRDER BEARING.
9. TILT PANEL SHALL BE NORMAL WEIGHT WITH f'_c = 4000 PSI MINIMUM AT 28 DAYS, UNLESS NOTED OTHERWISE.
10. REINFORCEMENT SHALL BE CONTINUOUS THROUGH KNOCKOUT AREA (FUTURE OPENING).
11. TRIM VERTICAL BARS BACK AT STEPPED EDGES, TYPICAL TOP OF TILT PANEL.
12. SEE PANEL ELEVATIONS FOR LOCATIONS OF PANEL EMBED PLATES.
13. ALL TILT PANELS ARE ASSUMED TO BE PAINTED AND/OR WATERPROOFED, SEE ARCHITECTURAL DRAWINGS. UNPAINTED/UNPROTECTED PANELS SHALL HAVE 3% AIR ENTRAINMENT. CONCRETE SHALL BE A MINIMUM OF 3000 PSI AT THE TIME OF ERECTION. (CONCRETE SLUMP SHALL NOT EXCEED 4 INCHES).
14. FLY ASH AND/OR SLAG SHALL NOT BE PERMITTED IN TILT PANELS.
15. TILT PANELS CONSTRUCTION IS TO CONFORM TO ACI 117-90. (STANDARD TOLERANCES FOR CONCRETE CONSTRUCTION AND MATERIALS).
16. REINFORCING STEEL SHALL CONFORM TO ASTM A615 GRADE 60. REINFORCING STEEL DETAILING, FABRICATION AND PLACEMENT SHALL CONFORM TO THE LATEST EDITION OF CRSI'S (MANUAL OF STANDARD PRACTICE). REINFORCING STEEL IS NOT TO BE SPLICED WITHOUT THE WRITTEN APPROVAL FROM THE STRUCTURAL ENGINEER.
17. SUBMIT SHOP DRAWINGS SHOWING ALL WORK RELATED TO TILT PANELS, INCLUDING TEMPORARY BRACING AND ERECTION PROCEDURES. REINFORCING BARS SHOWN ON THE CONTRACT DRAWINGS DO NOT ALLOW FOR LIFTING AND ERECTION STRESSES. WITH THE SHOP DRAWINGS SUBMIT DESCRIPTION COVERING ERECTION PROCEDURES, ADDITIONAL REINFORCEMENT, LIFTING HARDWARE, AND BRACING.
18. PANELS HAVE BEEN DESIGNED FOR IN PLACE STRESSES ONLY. THE GENERAL CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY BRACING, LIFTING HARDWARE, ADDITIONAL REINFORCING REQUIRED FOR LIFTING AND ERECTION PROCEDURES. REINFORCING BARS SHOWN ON CONTRACT DOCUMENTS DO NOT ALLOW FOR LIFTING AND ERECTION STRESSES.
19. PROVIDE CONTINUOUS NON-SHRINK GROUT UNDER ALL PANELS. CONTRACTOR TO GROUT UNDER ALL PANELS IMMEDIATELY AFTER PANELS ARE SET.
20. SHIM SHALL BE PLACED AT 5 FEET MAXIMUM (OR CLOSER) UNDER THE PANELS TO AVOID A CONTINUOUS FOOTING FROM BOWING UP DUE TO INVERSE SOIL PRESSURE.
21. KO INDICATES KNOCK OUT PANEL.
22. #3 LATERAL TIES SHALL BE BENT FROM STRAIGHT BARS (COIL RODS SHALL NOT BE PERMITTED) TO PREVENT BOWING OF TIES AND DECREASING CLEAR COVER.
23. SUBMIT SHOP DRAWINGS SHOWING ALL PANEL EMBED PLATES AND LOCATIONS. GENERAL CONTRACTOR SHALL COORDINATE.
24. SEE ARCHITECTURAL DRAWINGS AND DETAILS FOR PANEL JOINT AND REVEAL JOINT DETAILS NOT SHOWN ON STRUCTURAL DRAWINGS.
25. PANELS ARE TO BE CAST OUTSIDE FACE DOWN. CASTING SLAB SHALL BE CURED. SAW CUTS, CRACKS AND JOINTS IN THE CASTING SLAB SHALL BE FILLED AND LEVELED WITH A SEALANT SO AS TO MINIMIZE TRANSFER OF THE JOINT LINE TO THE PANEL FACE.
26. TILT PANEL CONTRACTOR SHALL BE RESPONSIBLE FOR THE COMPATIBILITY OF CURING AGENTS, SEALANTS AND RELEASING AGENTS UTILIZED IN THE WORK. IF PANELS ARE TO BE STACKED, THE TROWELED SURFACE SHALL BE CONSIDERED THE CASTING SLAB AND TREATED AS SUCH BONDBREAKER SHALL BE APPLIED IN ACCORDANCE WITH THE MANUFACTURERS REQUIREMENTS.
27. SEE PLAN FOR TILT PANEL THICKNESS.
28. CRACKS IN PANELS DUE TO LIFTING OR SHRINKAGE ARE TO BE REPAIRED BY THE GENERAL CONTRACTOR. PANELS THAT HAVE YIELDED AND CRACKED DURING LIFTING ARE TO BE REPLACED BY THE GENERAL CONTRACTOR.
29. GENERAL CONTRACTOR IS NOT TO DAMAGE FLOOR SLAB DURING PANEL ERECTION DUE TO CRANE LOADING. CONTRACTOR TO SUPPLY DESIGN ENGINEER WITH MAXIMUM CRANE LOADING ON SLAB FOR FLOOR STRENGTH EVALUATION PRIOR TO PANEL ERECTION. ANY DAMAGE TO THE FLOOR SLAB INCLUDING STRESS CRACKS RESULTING FROM LOADS EXCEEDING THOSE PROVIDED TO THE STRUCTURAL ENGINEER ARE TO BE REPAIRED BY THE GENERAL CONTRACTOR. REPLACEMENT OF SLAB WILL BE REQUIRED IN AREAS SEVERELY DAMAGED DURING LIFTING.
30. ALL TEMPORARY BRACING SHALL REMAIN IN PLACE UNTIL ROOF DECK IS FULLY INSTALLED AND INSPECTED, AND CONCRETE POUR BACK STRIP IS FULLY INSTALLED AND HAS REACHED A MINIMUM STRENGTH OF 3000 PSI.

N

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
OVERALL TILT PANEL PLAN

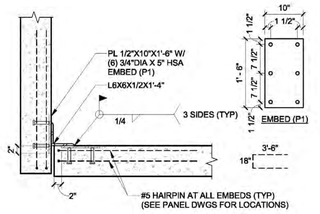
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S-300

CONSTRUCTION SET

9.119

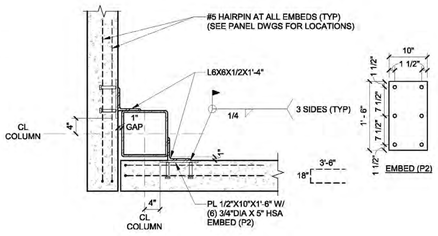
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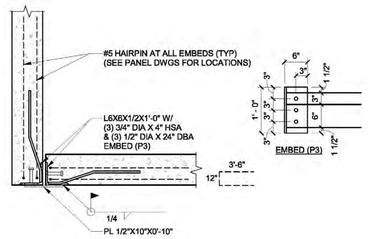
NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

1 / S-301 — TYPICAL TILT PANEL CONNECTION AT BUTT JOINT SECTION (P1)
SCALE: 3/4" = 1'-0"



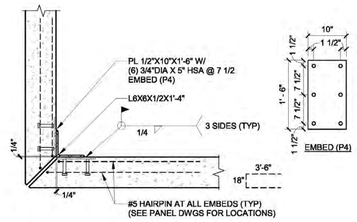
NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

2 / S-301 — TYPICAL TILT PANEL CONNECTION AT CORNER COLUMN SECTION (P2)
SCALE: 3/4" = 1'-0"



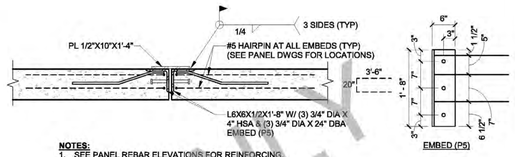
NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

3 / S-301 — TYPICAL TILT PANEL CONNECTION AT BUTT JOINT PANEL SECTION (P3)
SCALE: 3/4" = 1'-0"



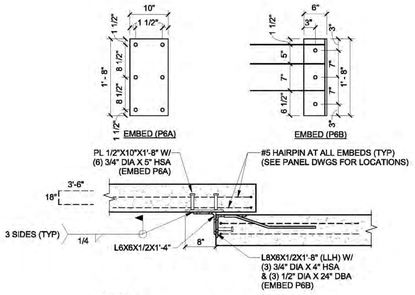
NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

4 / S-301 — TYPICAL TILT PANEL CONNECTION AT MITER JOINT PANEL SECTION (P4)
SCALE: 3/4" = 1'-0"



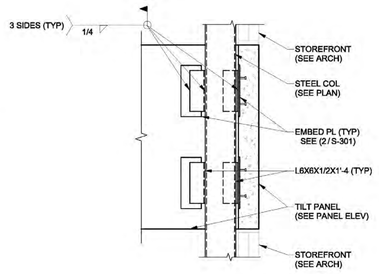
NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

5 / S-301 — TYPICAL TILT PANEL TO TILT PANEL CONNECTION SECTION (P5)
SCALE: 3/4" = 1'-0"

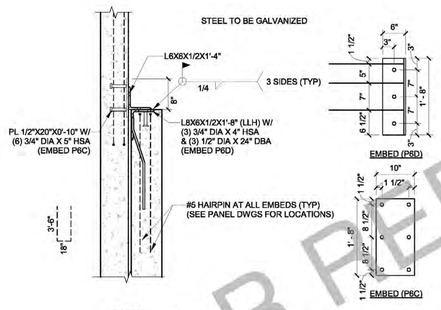


NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

6 / S-301 — TYPICAL TILT PANEL CONNECTION AT OVERLAPPING PANELS SECTION (P6A) & (P6B)
SCALE: 3/4" = 1'-0"

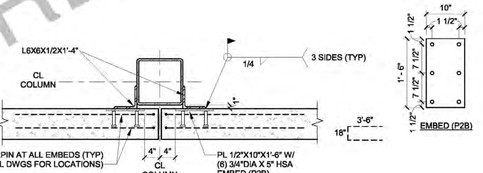


7 / S-301 — TYPICAL AT SPANDREL TILT PANEL TO HSS COLUMN CONNECTION SECTION (P8)
SCALE: 1/2" = 1'-0"



NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

8 / S-301 — TYPICAL TILT PANEL CONNECTION AT OVERLAPPING PANELS SECTION (P6C) & (P6D)
SCALE: 3/4" = 1'-0"

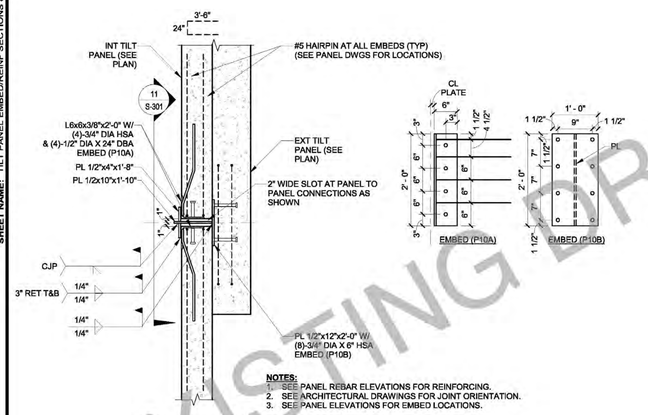


NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

9 / S-301 — TYPICAL TILT PANEL CONNECTION AT CORNER COLUMN SECTION (P2B)
SCALE: 3/4" = 1'-0"

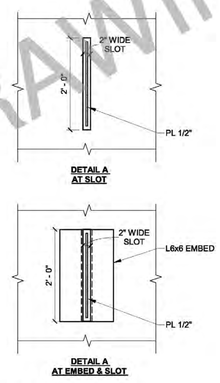
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B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
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S15	13	S-540
S16 & S16B	3	S-541

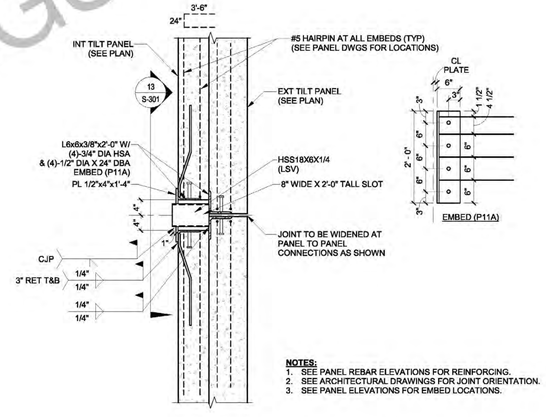


NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

10 / S-301 — TYPICAL TILT PANEL CONNECTION AT THE END OF A SANDWHICH JOINT (P10A) & (P10B)
SCALE: 3/4" = 1'-0"

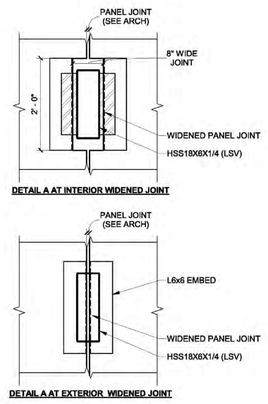


11 / S-301 — SLOT DETAIL
SCALE: 3/4" = 1'-0"



NOTES:
1. SEE PANEL REBAR ELEVATIONS FOR REINFORCING.
2. SEE ARCHITECTURAL DRAWINGS FOR JOINT ORIENTATION.
3. SEE PANEL ELEVATIONS FOR EMBED LOCATIONS.

12 / S-301 — TYPICAL TILT PANEL CONNECTION AT SANDWHICH JOINT (P11A) & (P11B)
SCALE: 3/4" = 1'-0"



13 / S-301 — DETAIL AT WIDENED JOINT
SCALE: 3/4" = 1'-0"

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

2

Date 2024 JUNE 28
Job No. 24.0000849.000

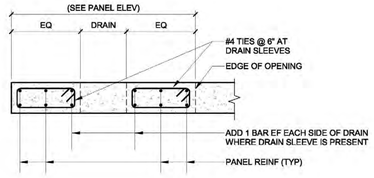
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TILT PANEL EMBED/REINF SECTIONS & DETAILS

Sheet No.
S-301

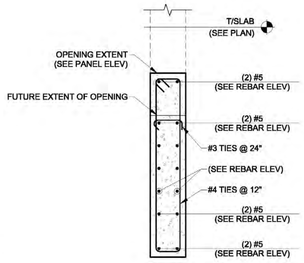
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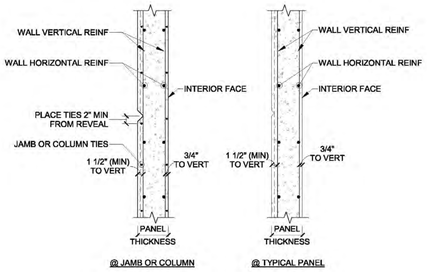
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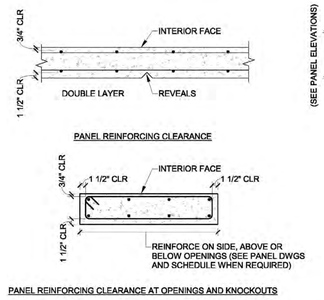
1 S-302 TYPICAL TILT PANEL REINFORCEMENT AT DRAIN DETAIL
SCALE: 3/4" = 1'-0"



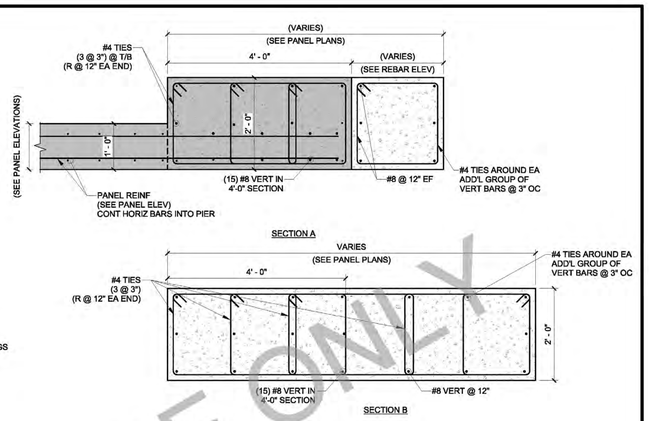
2 S-302 TYPICAL TILT PANEL REINFORCEMENT AT BOTTOM LOADING DOCK SECTION
SCALE: 3/4" = 1'-0"



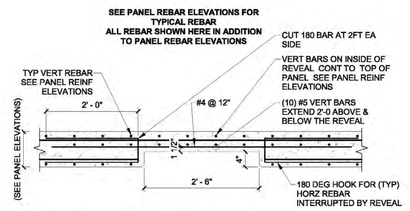
3 S-302 TYPICAL TILT PANEL REINFORCEMENT LAYOUT DETAIL
SCALE: NO SCALE



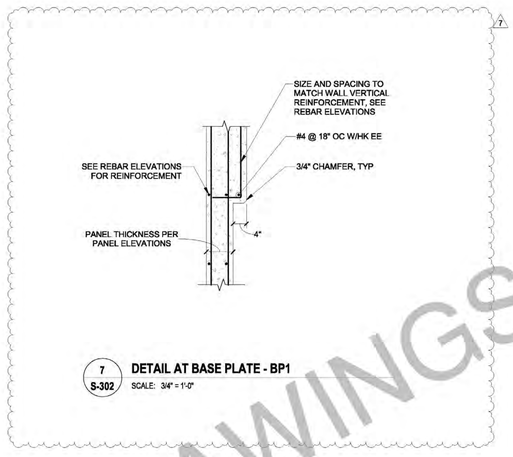
4 S-302 TYPICAL TILT PANEL REINFORCEMENT LAYOUT DETAIL
SCALE: 3/4" = 1'-0"



5 S-302 TILT PANEL REINFORCEMENT AT ENTRANCE PANEL
SCALE: 3/4" = 1'-0"



6 S-302 TILT PANEL REINFORCEMENT AT REVEAL
SCALE: 3/4" = 1'-0"



7 S-302 DETAIL AT BASE PLATE - BP1
SCALE: 3/4" = 1'-0"

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AUBURNDALE, FL 33823

Print Record

Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

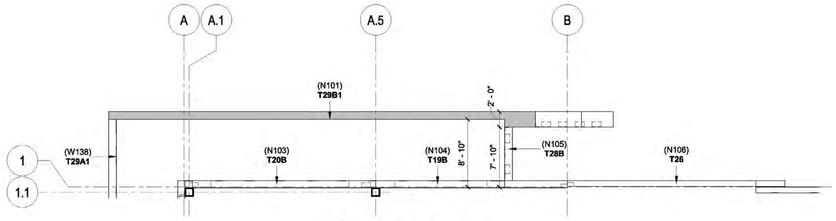
Sheet Title
TILT PANEL EMBED/REINF SECTIONS & DETAILS

Sheet No.
S-302 2

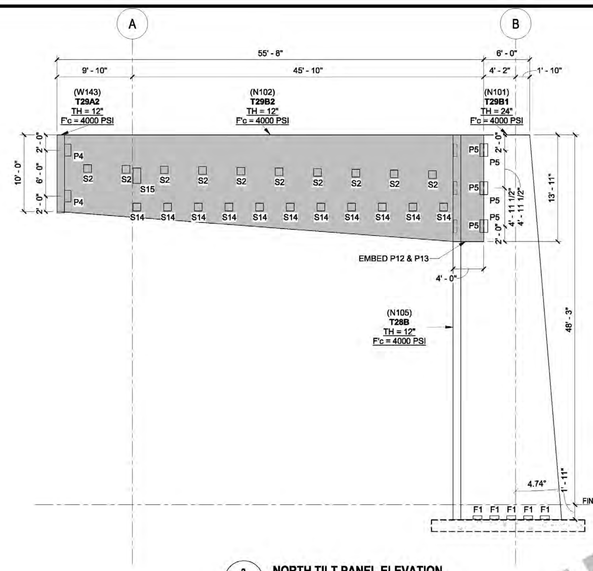
CONSTRUCTION SET

9.121

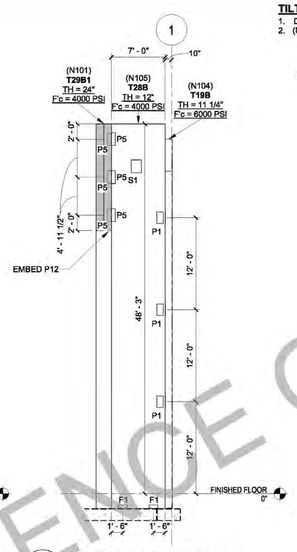
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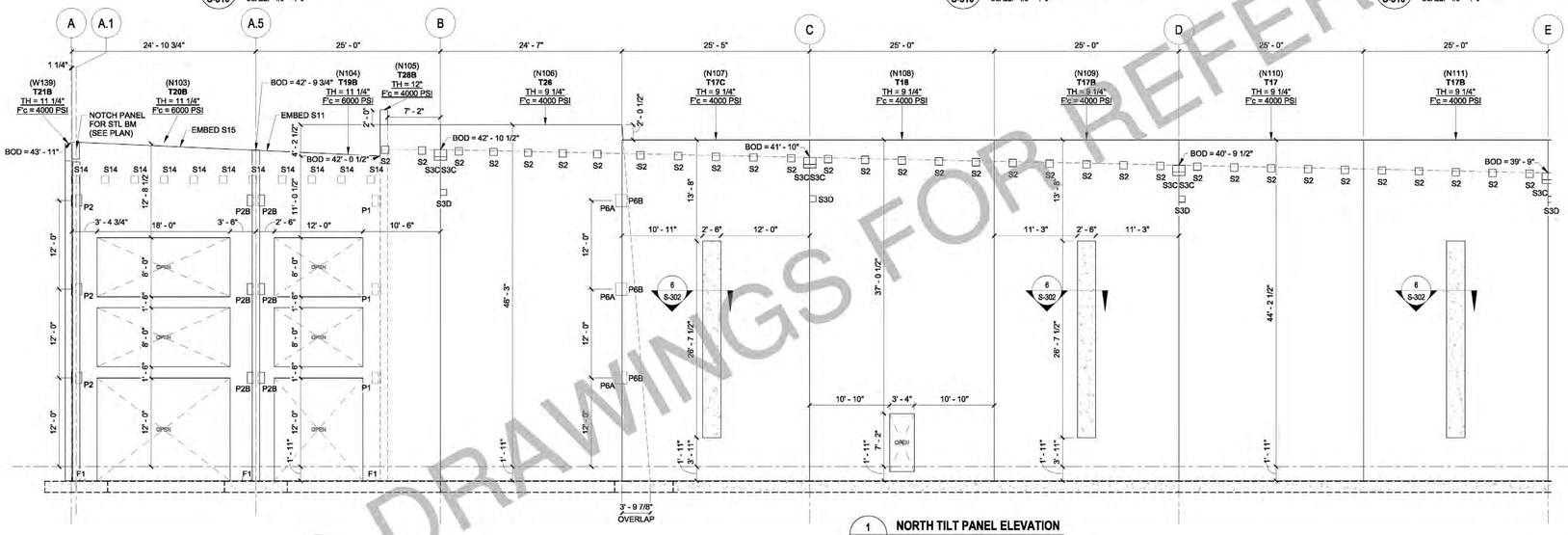
5 S-310 **PARTIAL PANEL PLAN** SCALE: 1/8" = 1'-0"



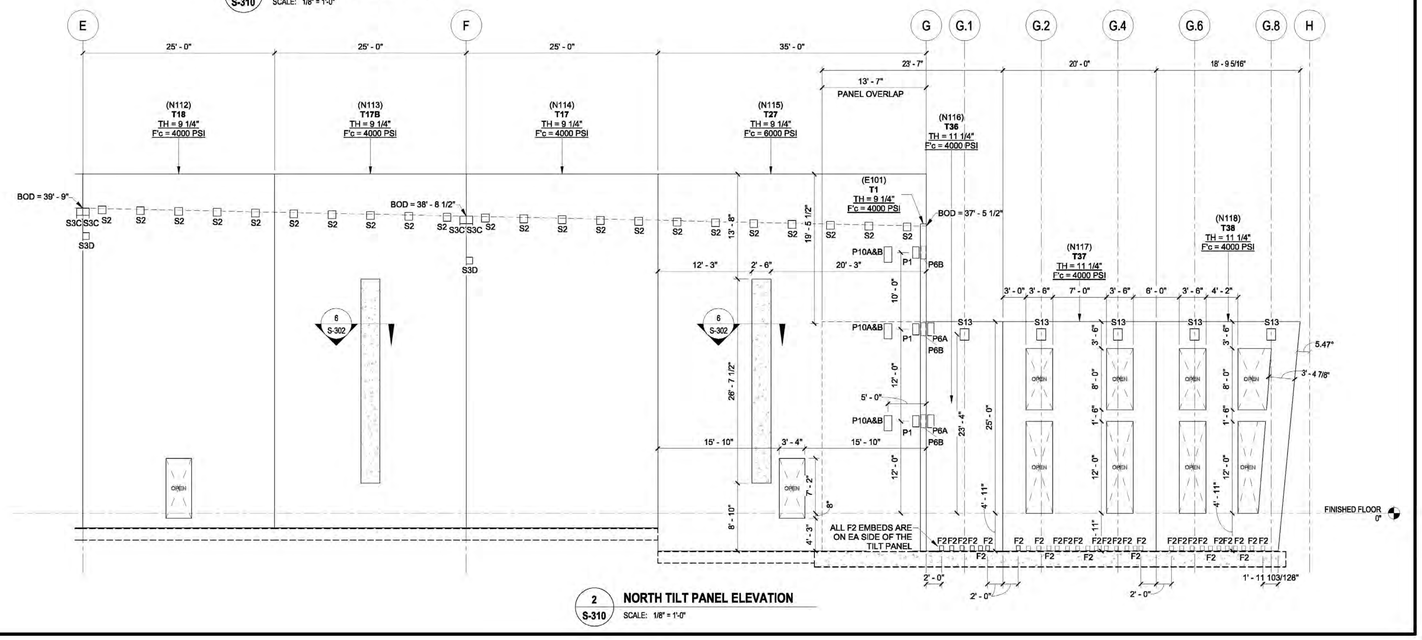
3 S-310 **NORTH TILT PANEL ELEVATION** SCALE: 1/8" = 1'-0"



4 S-310 **NORTH TILT PANEL ELEVATION** SCALE: 1/8" = 1'-0"



1 S-310 **NORTH TILT PANEL ELEVATION** SCALE: 1/8" = 1'-0"



2 S-310 **NORTH TILT PANEL ELEVATION** SCALE: 1/8" = 1'-0"

TILT PANEL ELEVATION LEGEND:

1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) — PANEL NUMBER (SEE ARCH)
   T1 — PANEL TYPE (SEE S-320 SERIES)
   TH — PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADD'L INFO)
   F'c = — COMPRESSIVE STRENGTH

EMBED SCHEDULE

PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541

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Print Record

Revisions

2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
NORTH TILT PANEL ELEVATIONS

Sheet No.
S-310 2

CONSTRUCTION SET

9.122

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PLOTTED: 2/28/2025 3:28:25 PM

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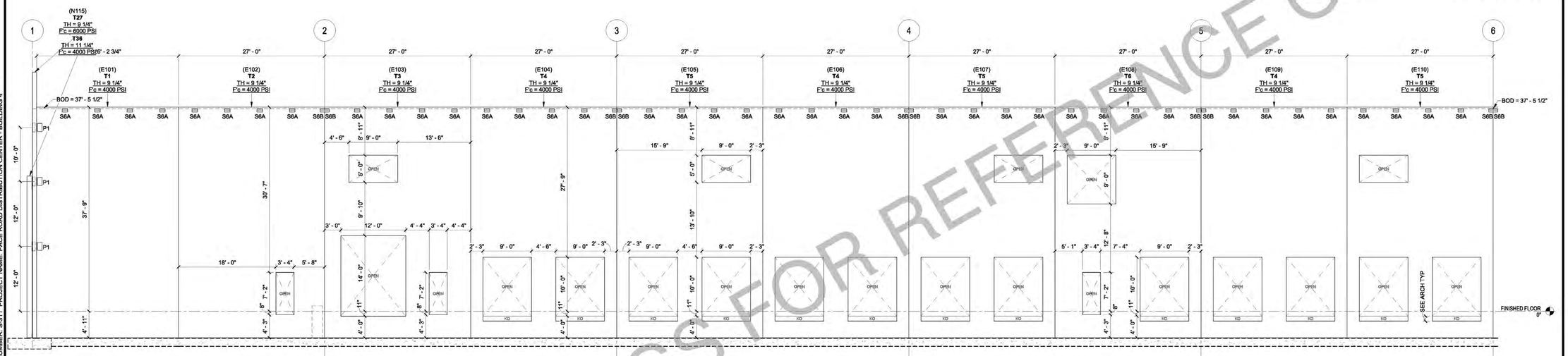
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TILT PANEL ELEVATION LEGEND:

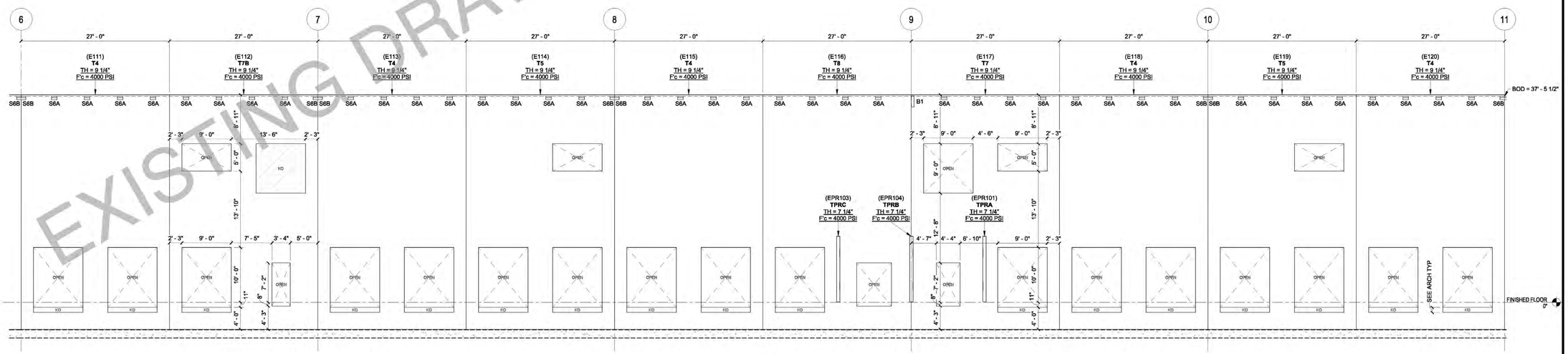
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) ◄ PANEL NUMBER (SEE ARCH)
   T1 ◄ PANEL TYPE (SEE S-320 SERIES)
   TH ◄ PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADD'L INFO)
   F'c = ◄ COMPRESSIVE STRENGTH

EMBED SCHEDULE

PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



1 S-311 **EAST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



2 S-311 **EAST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

EXISTING DRAWINGS FOR REFERENCE ONLY

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Print Record

Revisions

2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000R49.000

Sheet Title
EAST TILT PANEL ELEVATIONS

Sheet No.
S-311 2

CONSTRUCTION SET

9.123

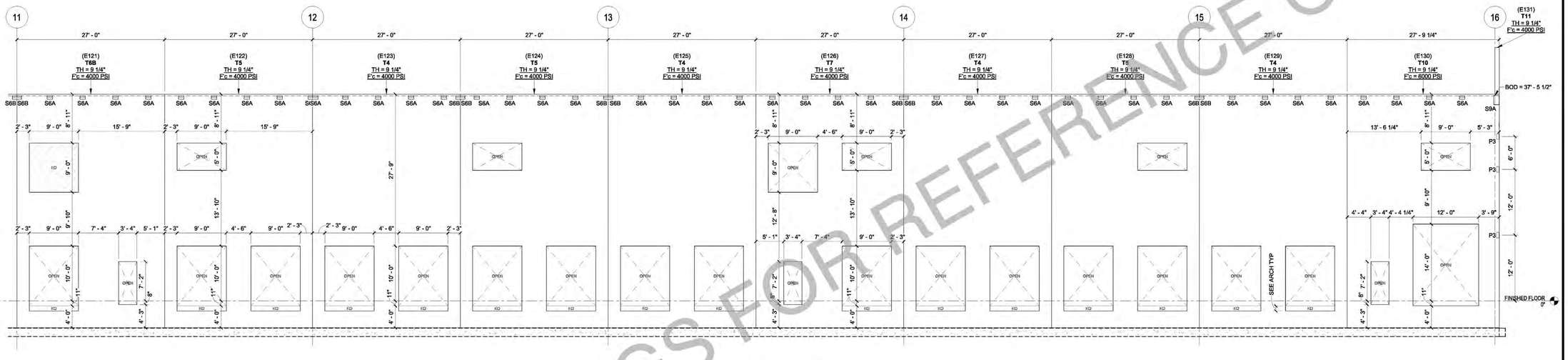
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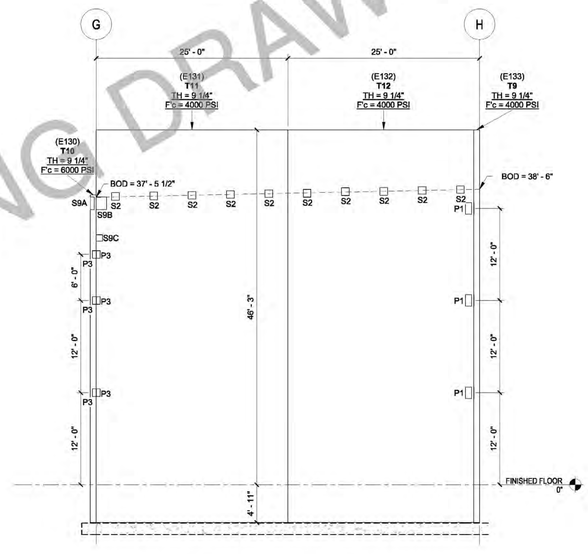
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) ◄ PANEL NUMBER (SEE ARCH)
   T1 ◄ PANEL TYPE (SEE S-320 SERIES)
   TH ◄ PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADD'L INFO)
   F'c = ◄ COMPRESSIVE STRENGTH

EMBED SCHEDULE

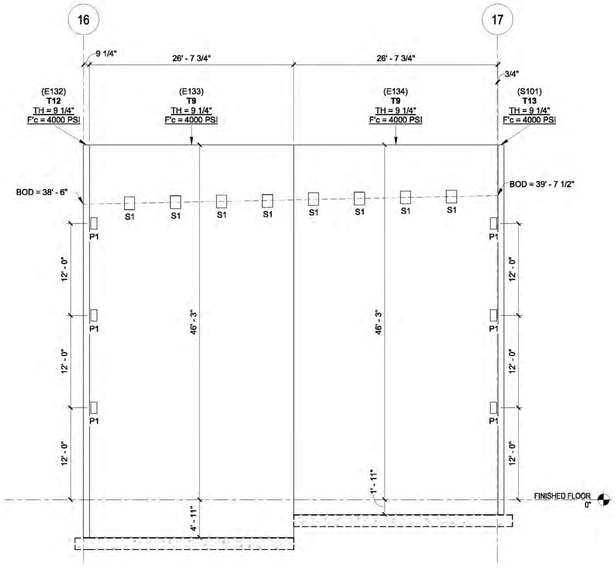
PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



1 S-312 **EAST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



2 S-312 **EAST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



3 S-312 **EAST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
EAST TILT PANEL ELEVATIONS

Sheet No.
S-312 2

CONSTRUCTION SET

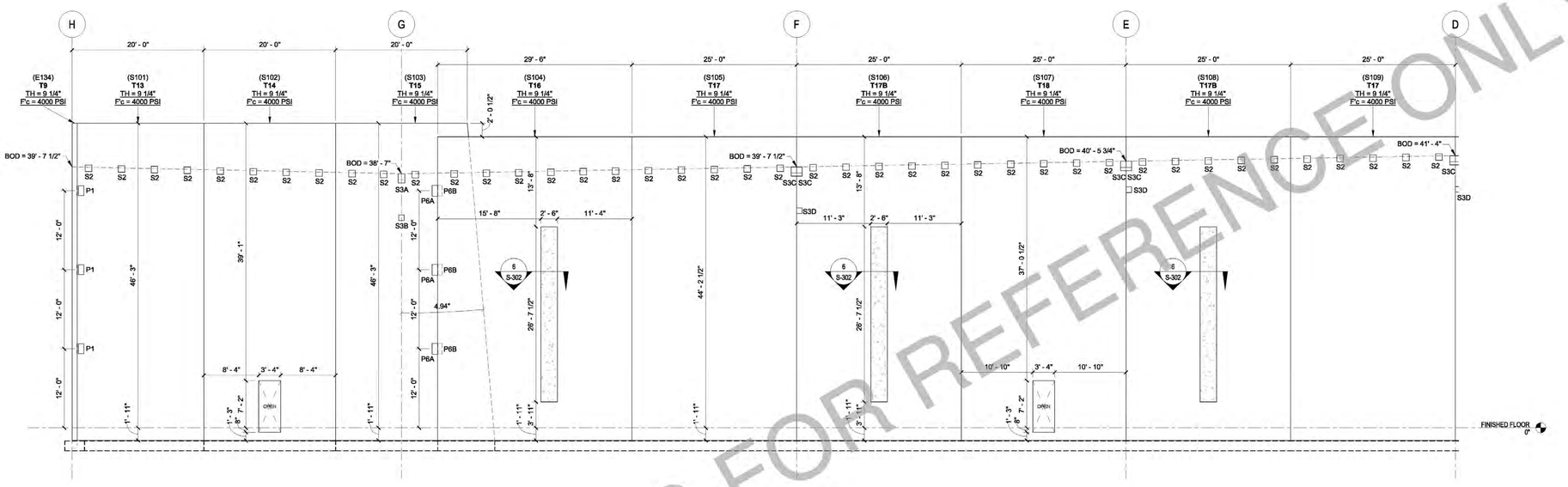
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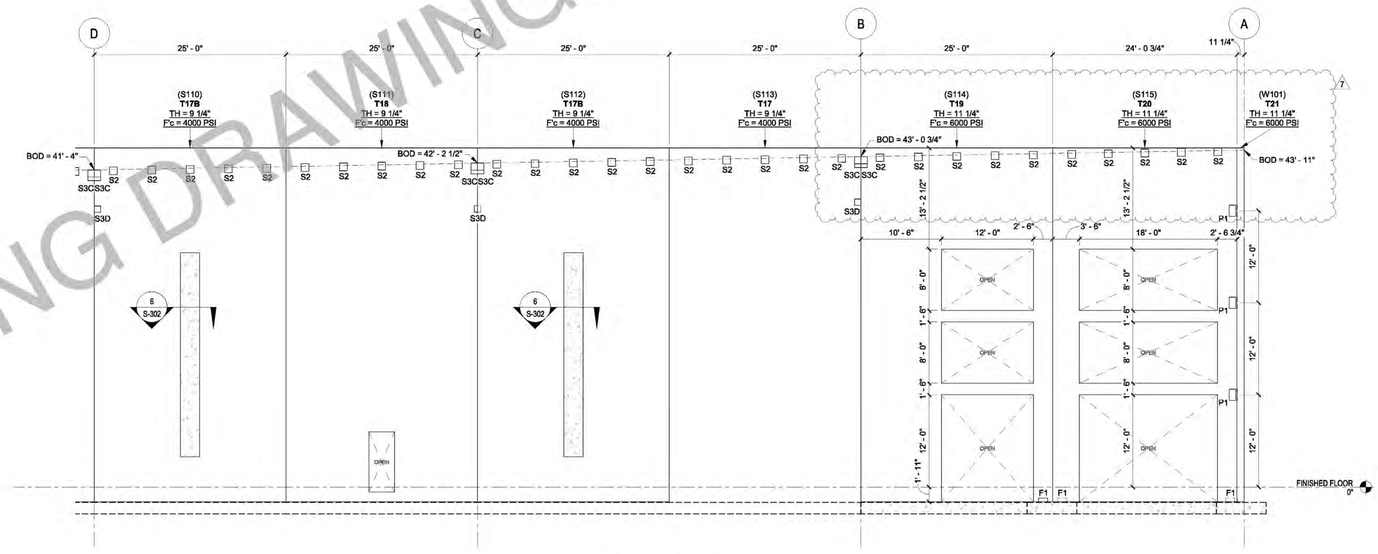
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) — PANEL NUMBER (SEE ARCH)
   T1 — PANEL TYPE (SEE S-320 SERIES)
   TH — PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADD'L INFO)
   F'c = — COMPRESSIVE STRENGTH

EMBED SCHEDULE

PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



1 S-313 **SOUTH TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



2 S-313 **SOUTH TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET
7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
SOUTH TILT PANEL ELEVATIONS

Sheet No.
S-313 2

CONSTRUCTION SET

9.125

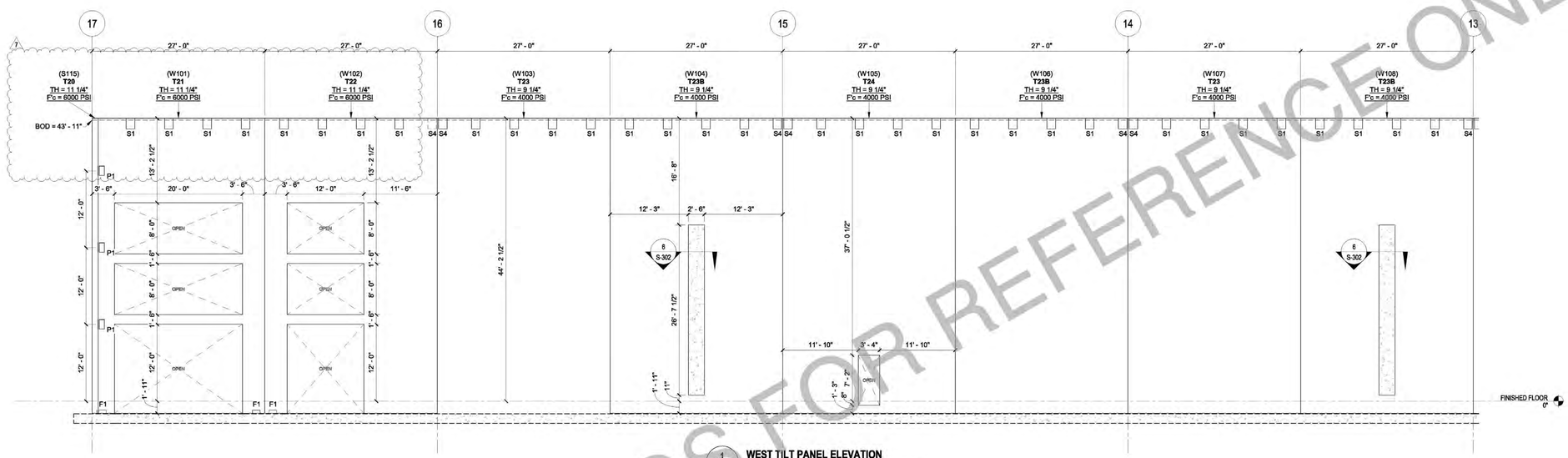
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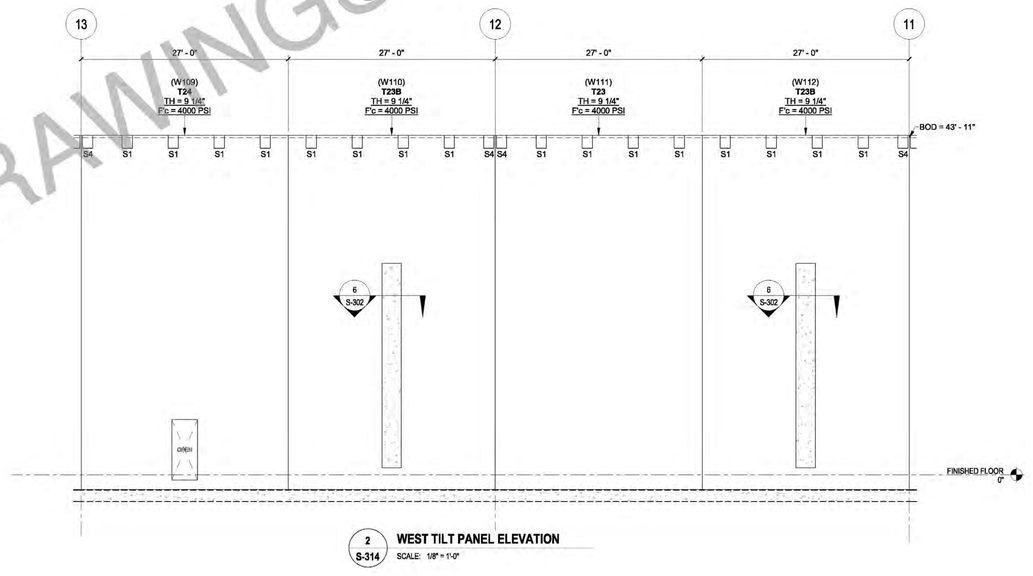
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) ◄ PANEL NUMBER (SEE ARCH)
   T1 ◄ PANEL TYPE (SEE S-320 SERIES)
   TH ◄ PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADD'L INFO)
   F'c = ◄ COMPRESSIVE STRENGTH

EMBED SCHEDULE

PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



1 S-314 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



2 S-314 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET
7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
WEST TILT PANEL ELEVATIONS

Sheet No.
S-314 2

CONSTRUCTION SET

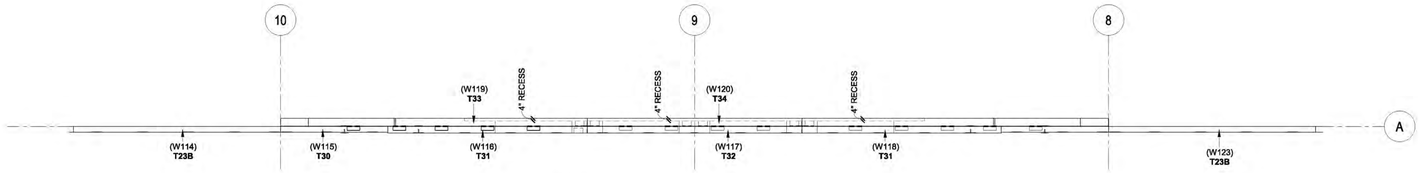
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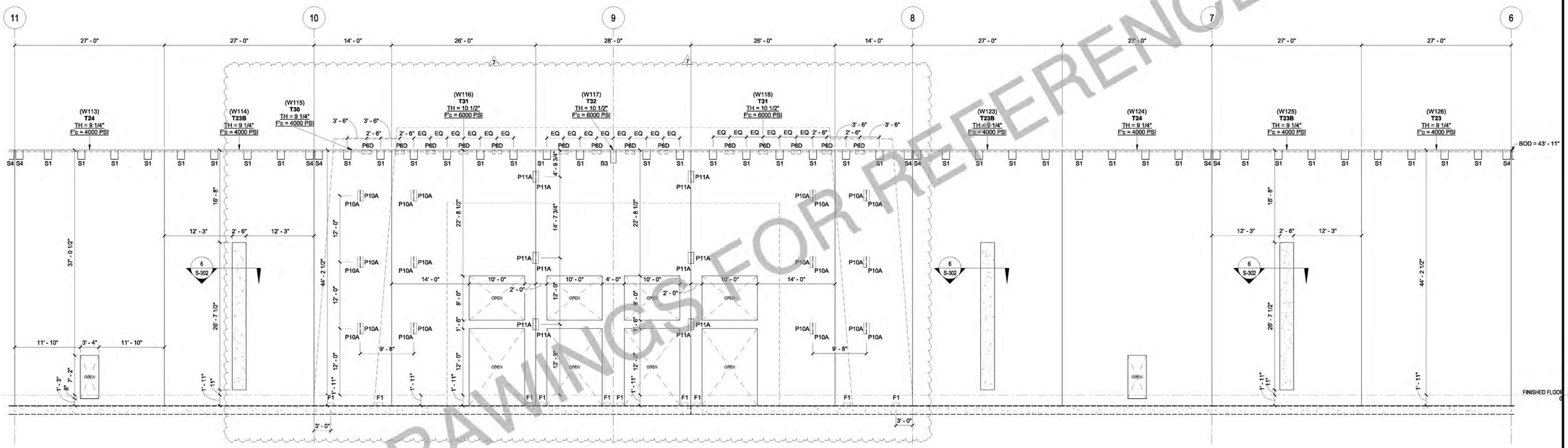
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) ← PANEL NUMBER (SEE ARCH)
   T1 ← PANEL TYPE (SEE S-320 SERIES)
   TH ← PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADDL INFO)
   F'c = ← COMPRESSIVE STRENGTH

EMBED SCHEDULE

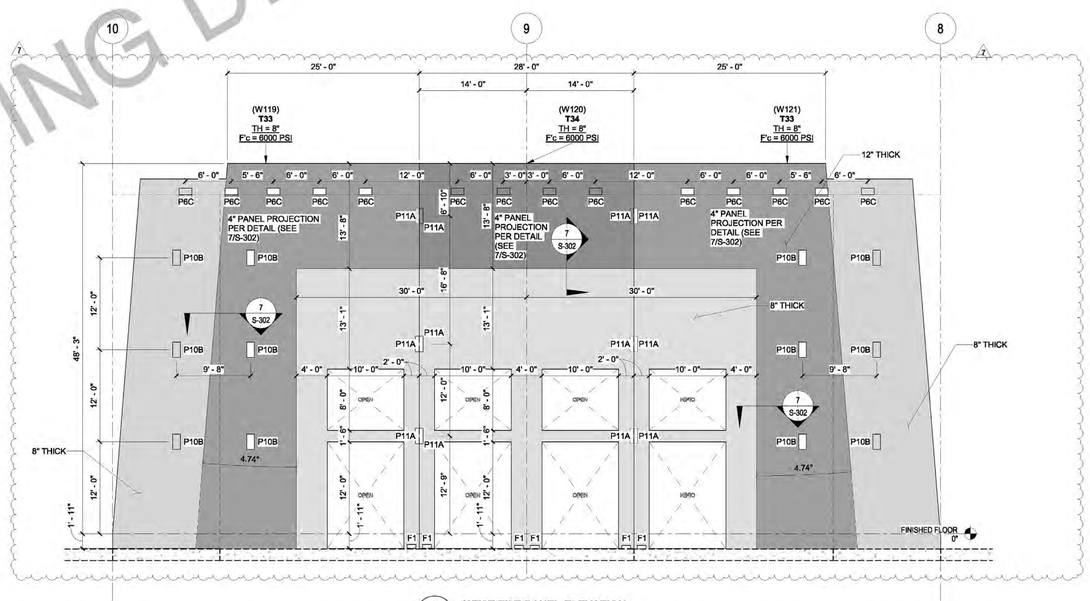
PLATE NAME	DETAIL NUMBER	SHEET NUMBER
B1	8	S-530
B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



3 S-315 **PARTIAL PANEL PLAN**
SCALE: 1/8" = 1'-0"



1 S-315 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



2 S-315 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

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Print Record

Revisions

2	2024 JUNE 28	CONSTRUCTION SET
7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
WEST TILT PANEL ELEVATIONS

Sheet No.
S-315 2

CONSTRUCTION SET

9.127

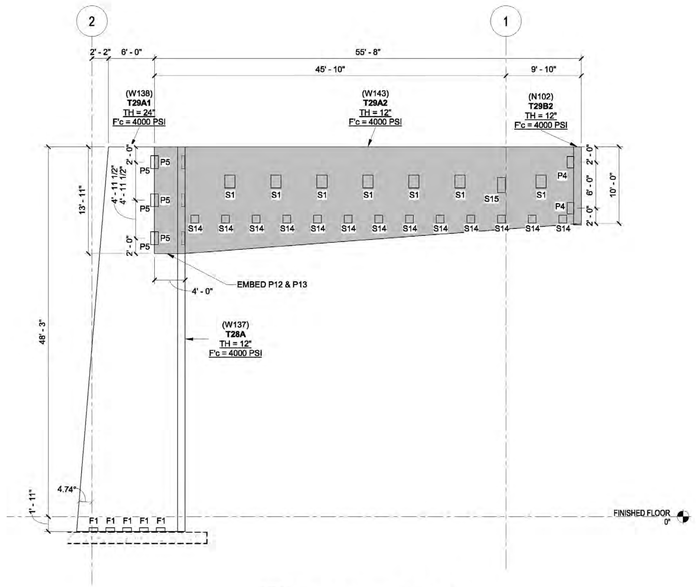
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TILT PANEL ELEVATION LEGEND:

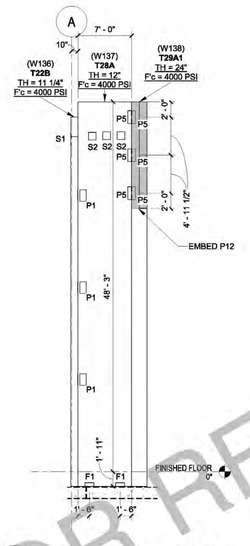
1. DIMENSIONS TO VERTICAL EMBEDS ARE (TYP-UNO)
2. (N-101) ◄ PANEL NUMBER (SEE ARCH)
   T1 ◄ PANEL TYPE (SEE S-320 SERIES)
   TH ◄ PANEL THICKNESS (SEE S-301 & S-320 SERIES FOR ADDL INFO)
   F'c = ◄ COMPRESSIVE STRENGTH

EMBED SCHEDULE

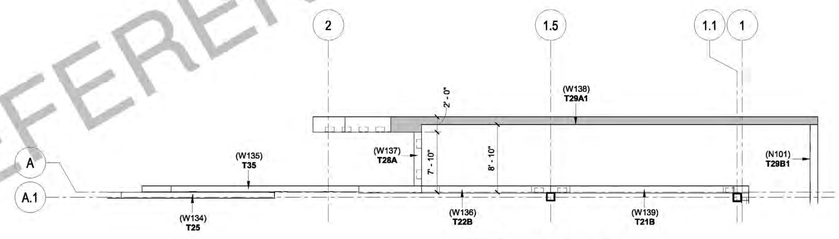
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B1	9	S-530
B3	11	S-530
F1	6	S-501
F2	1	S-540
P1	1	S-301
P2	2	S-301
P3	3	S-301
P4	4	S-301
P5	5	S-301
P6A & P6B	6	S-301
P6C & P6D	8	S-301
P8	7	S-301
P10A & P10B	10	S-301
P11A & P11B	12	S-301
P12 & P13	6	S-541
S1	6	S-520
S2	7	S-520
S3A	10	S-520
S3A	11	S-520
S3A & S3B & S3C	1	S-521
S4	12	S-520
S6A & S6B	8	S-520
S9A & S9B & S9C	2	S-521
S12 & S12B	2	S-541
S13	4	S-541
S14	5	S-541
S15	13	S-540
S16 & S16B	3	S-541



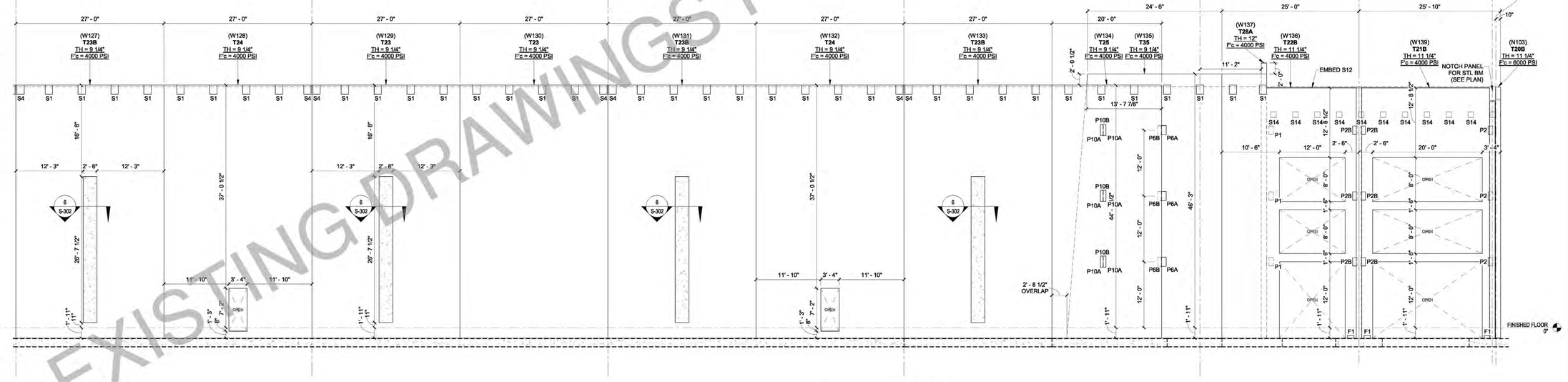
2 S-316 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



3 S-316 **SOUTH TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"



4 S-316 **PARTIAL PANEL PLAN**
SCALE: 1/8" = 1'-0"



1 S-316 **WEST TILT PANEL ELEVATION**
SCALE: 1/8" = 1'-0"

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2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
WEST TILT PANEL ELEVATIONS

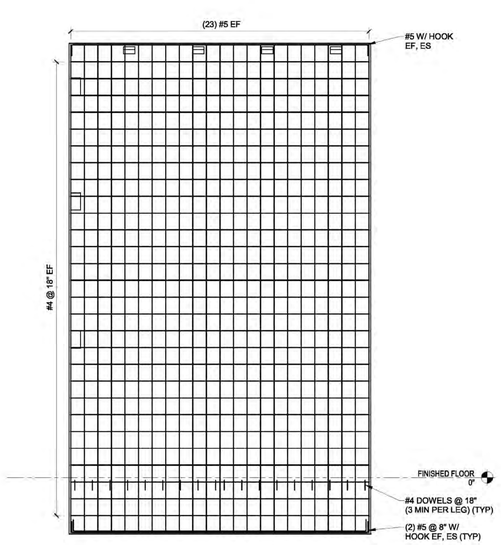
Sheet No.
S-316 2

CONSTRUCTION SET

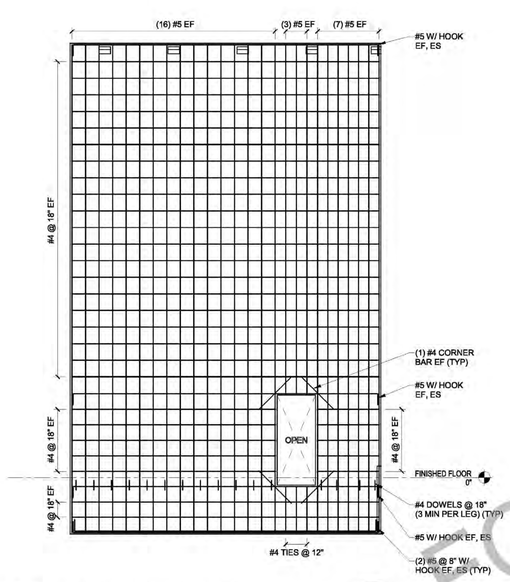
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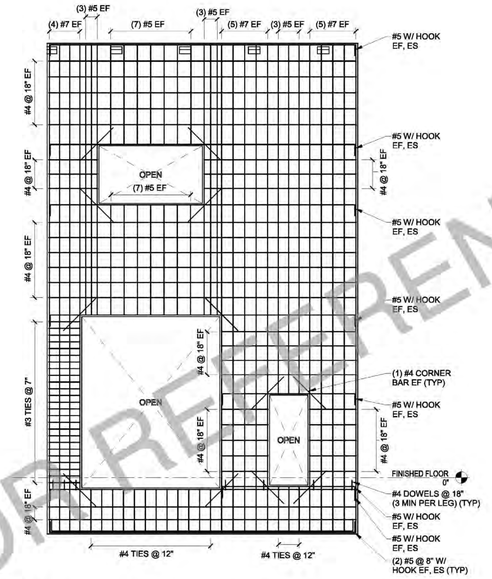
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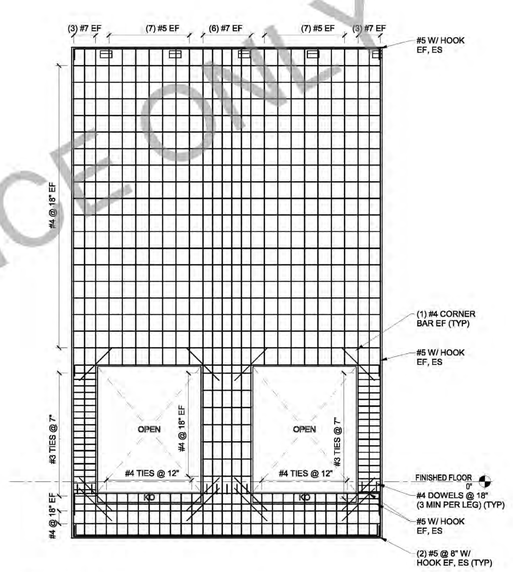
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SCALE: 3/16" = 1'-0"



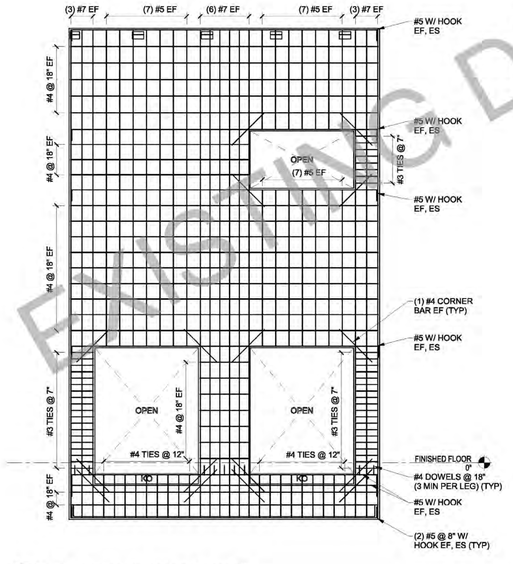
2 S-320 **T2 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



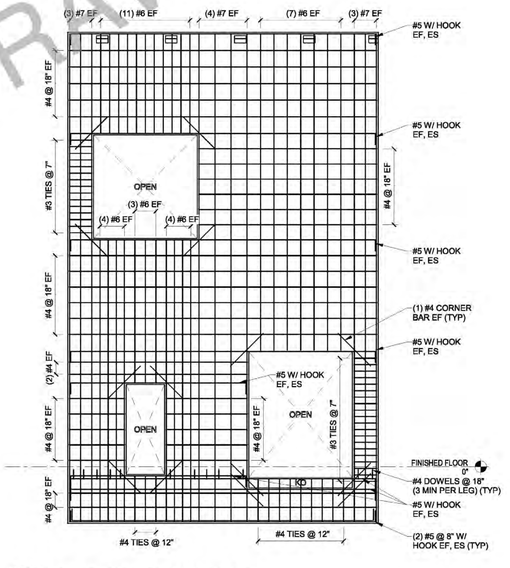
3 S-320 **T3 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



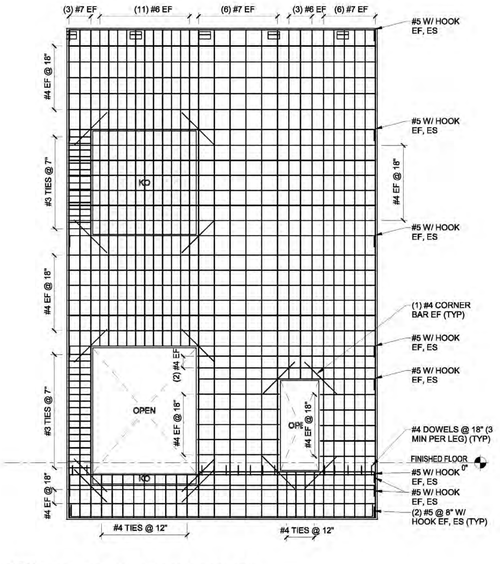
4 S-320 **T4 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



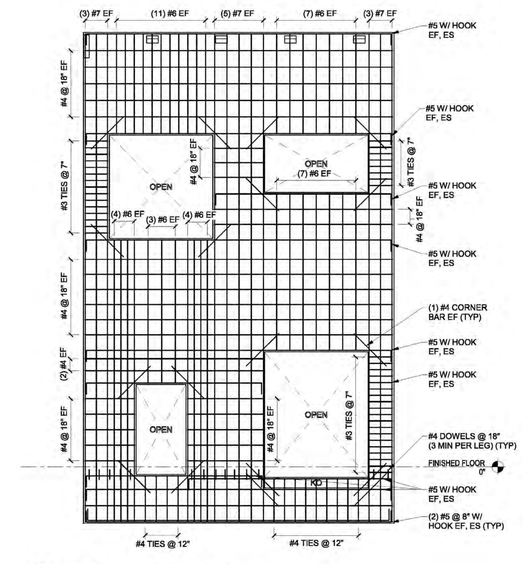
5 S-320 **T5 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



6 S-320 **T6 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-320 **T6B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



8 S-320 **T7 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Revisions

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Date 2024 JUNE 28
Job No. 24.0000849.000

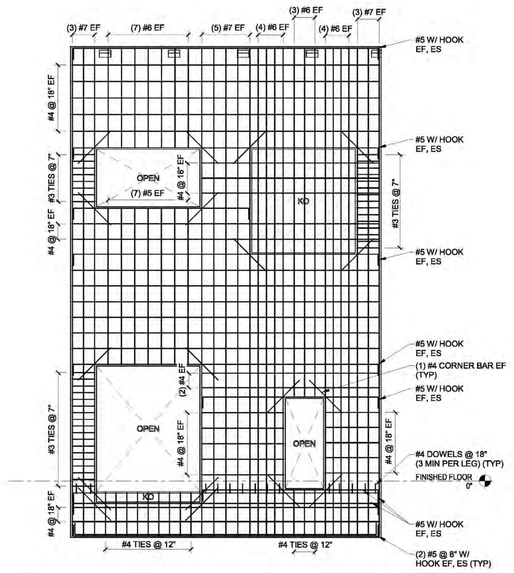
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TILT PANEL REBAR ELEVATIONS

Sheet No.
S-320 2

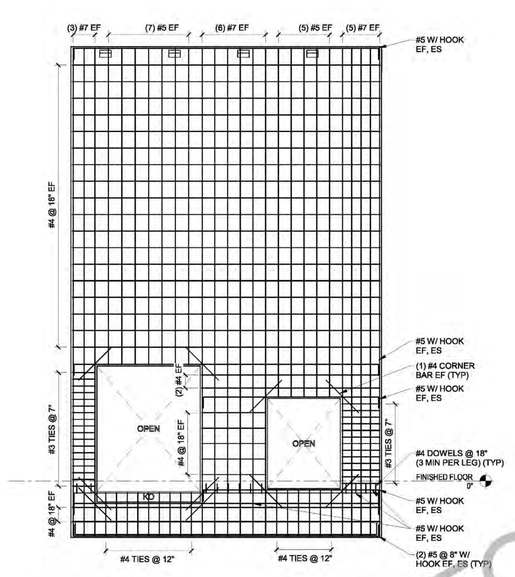
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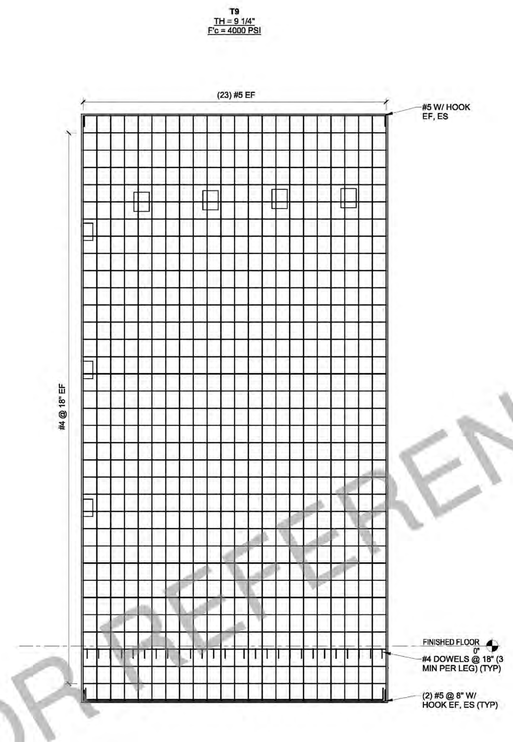
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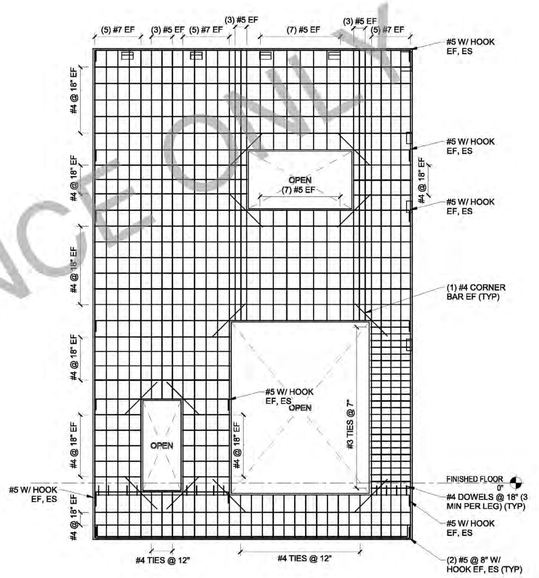
1 S-321 **T7B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



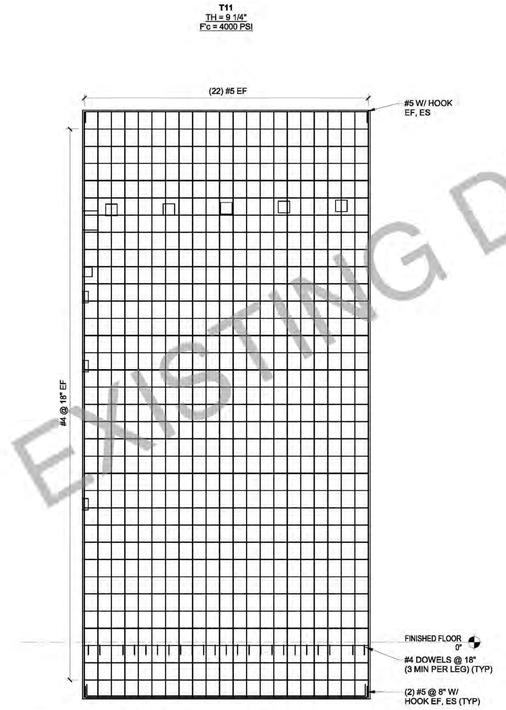
2 S-321 **T8 - TILT PANEL REBAR ELEVATION**
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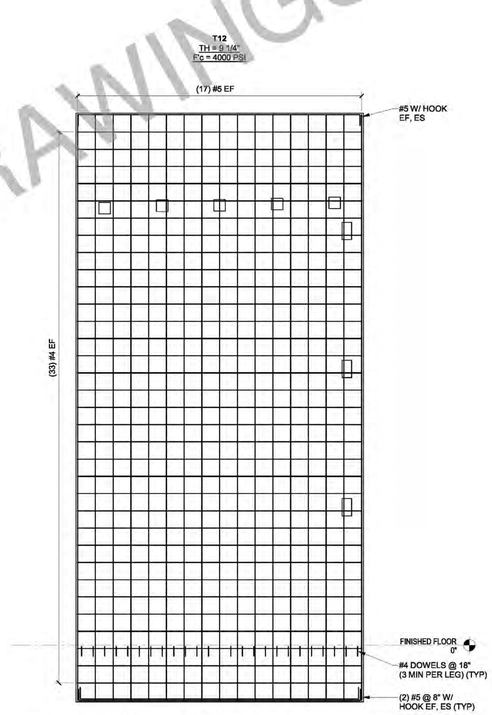
3 S-321 **T9 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



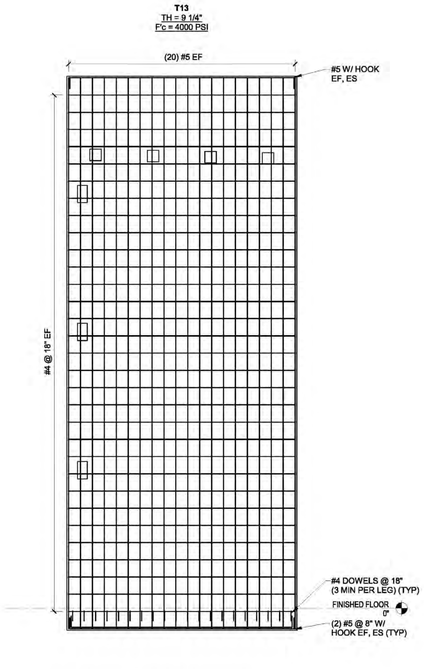
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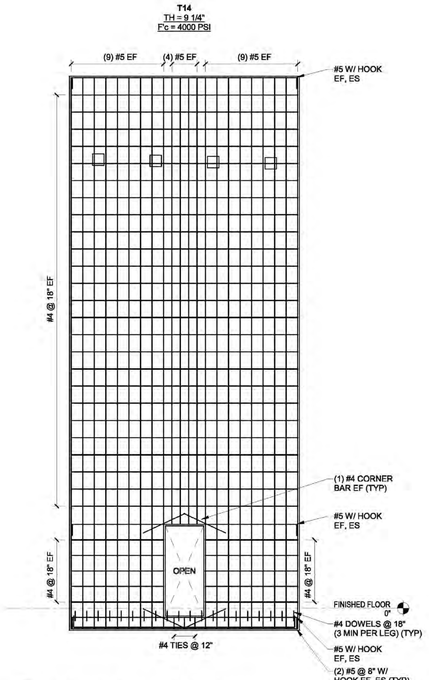
5 S-321 **T11 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



6 S-321 **T11 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-321 **T13 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



8 S-321 **T14 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.00008HS.000

Sheet Title
TILT PANEL REBAR ELEVATIONS

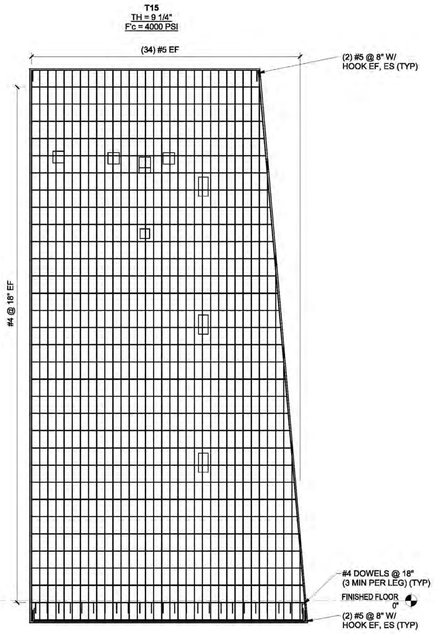
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S-321 2

CONSTRUCTION SET

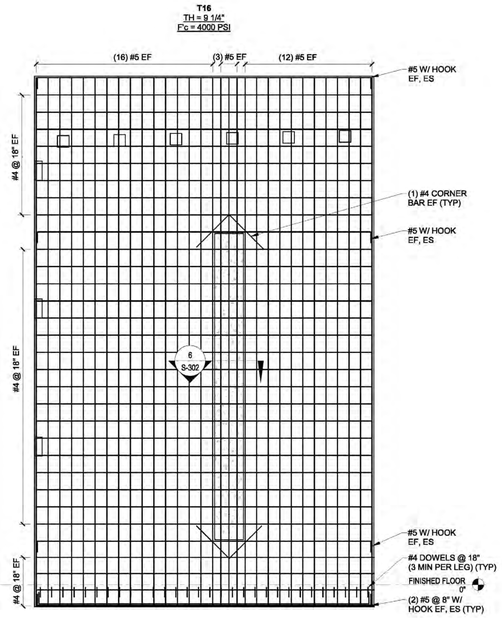
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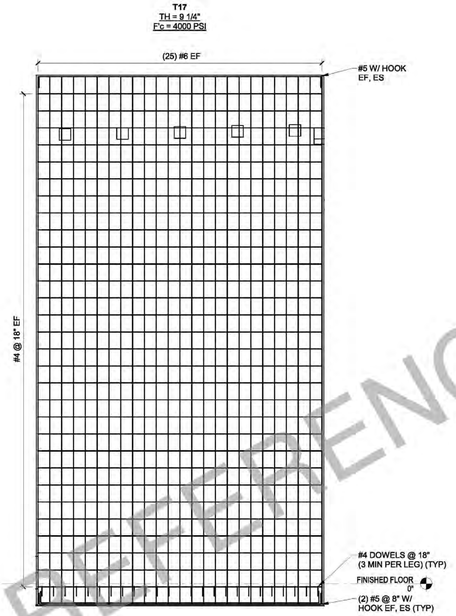
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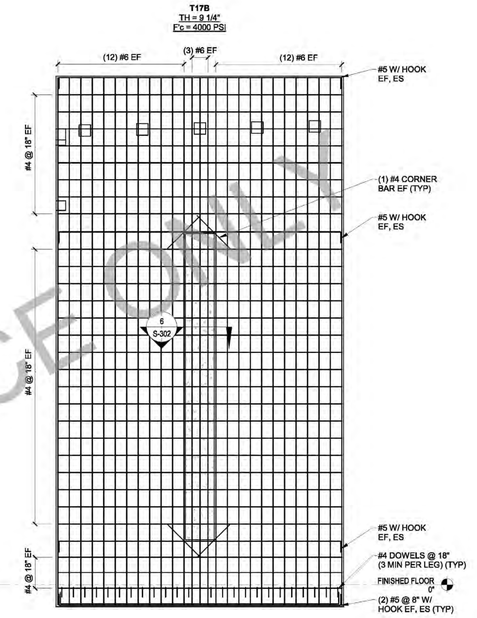
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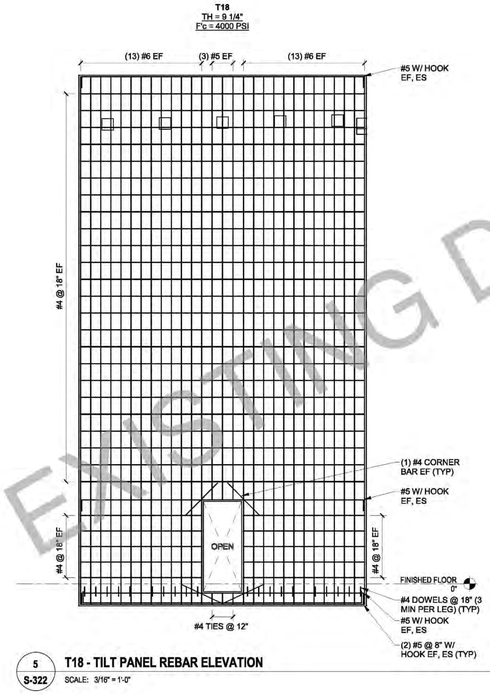
2 S-322 **T16 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



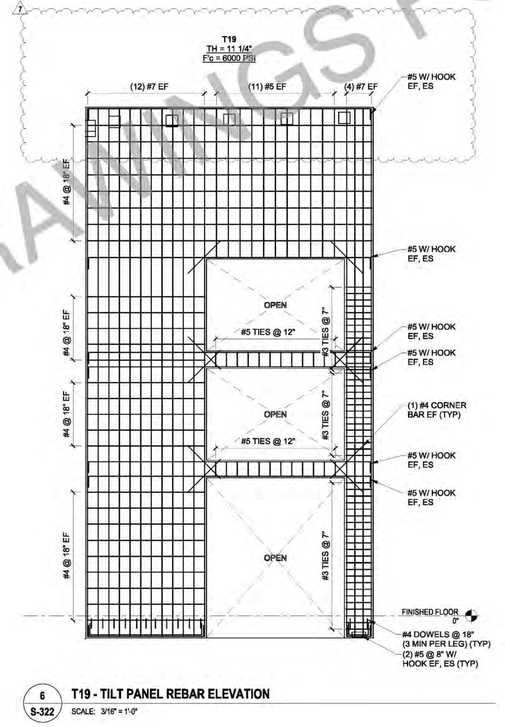
3 S-322 **T17 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



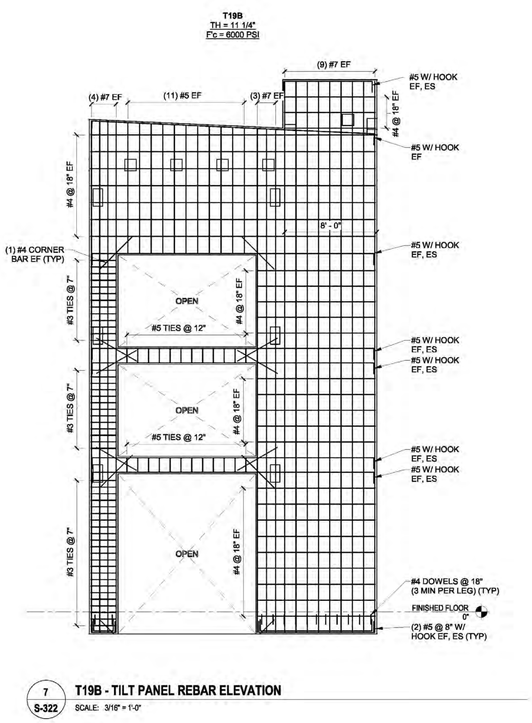
4 S-322 **T17B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



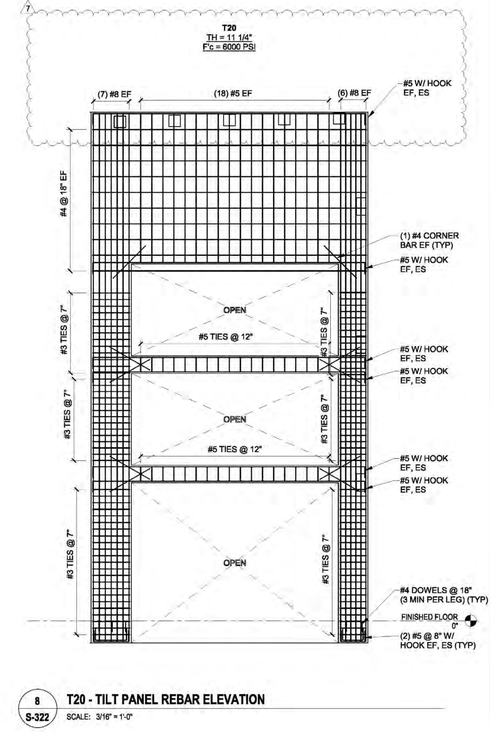
5 S-322 **T18 - TILT PANEL REBAR ELEVATION**
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6 S-322 **T19 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-322 **T19B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



8 S-322 **T20 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
TILT PANEL REBAR ELEVATIONS

Sheet No.
S-322 2

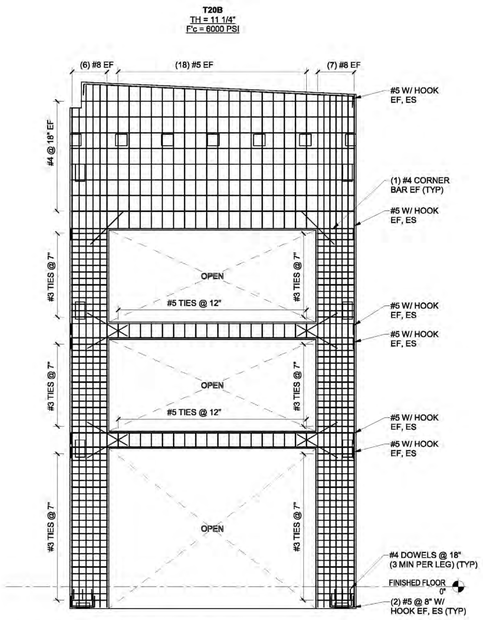
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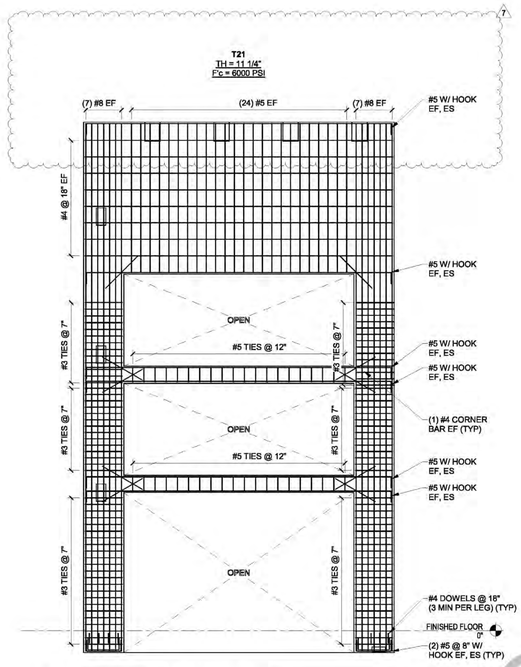
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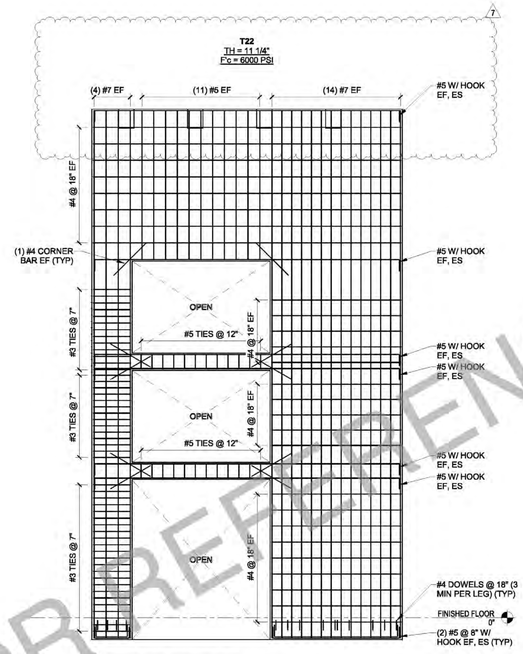
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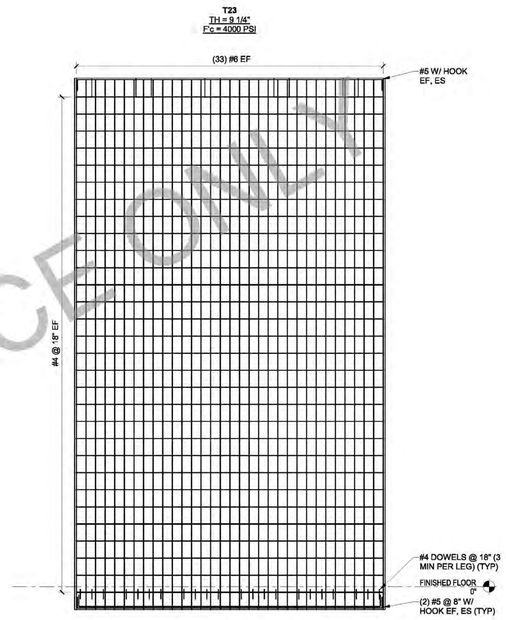
1 S-323 **T20B - TILT PANEL REBAR ELEVATION**
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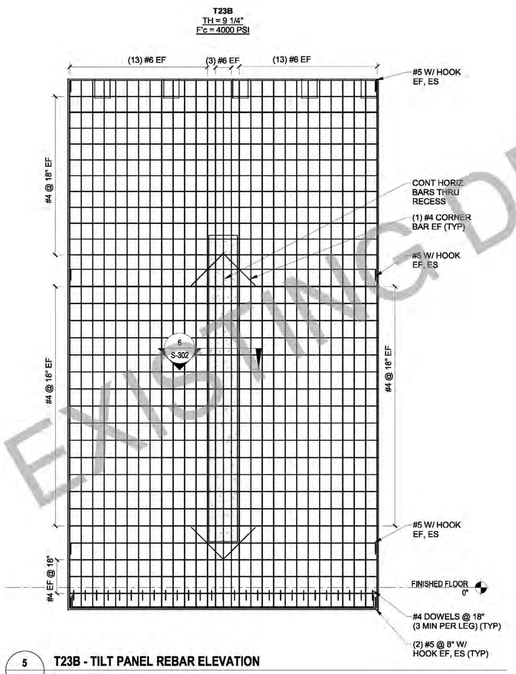
2 S-323 **T21 - TILT PANEL REBAR ELEVATION**
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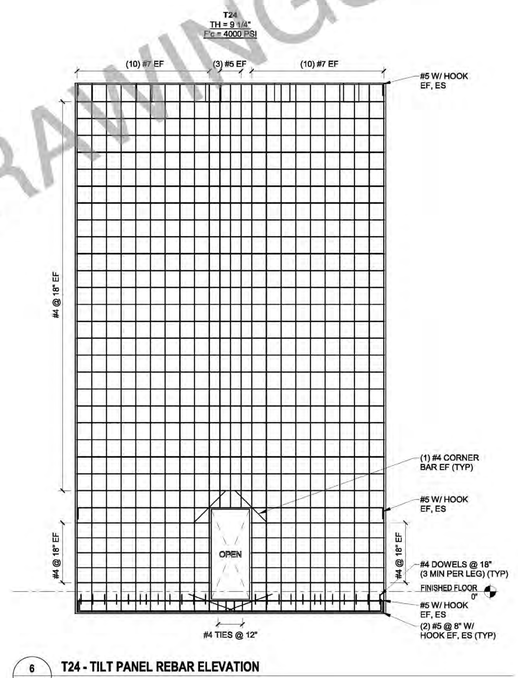
3 S-323 **T22 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



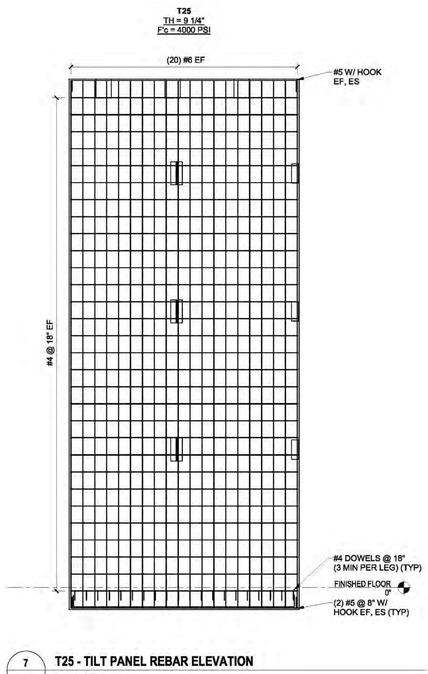
4 S-323 **T23 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



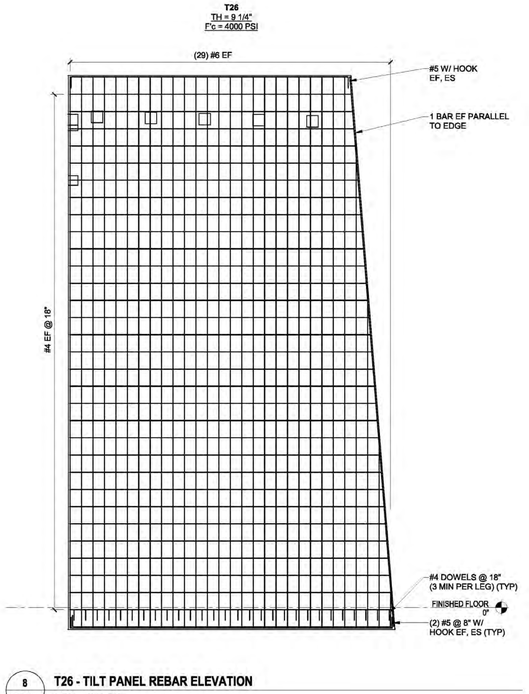
5 S-323 **T23B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



6 S-323 **T24 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-323 **T25 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



8 S-323 **T26 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET
7	2025 FEBRUARY 28	RFI REPONSE

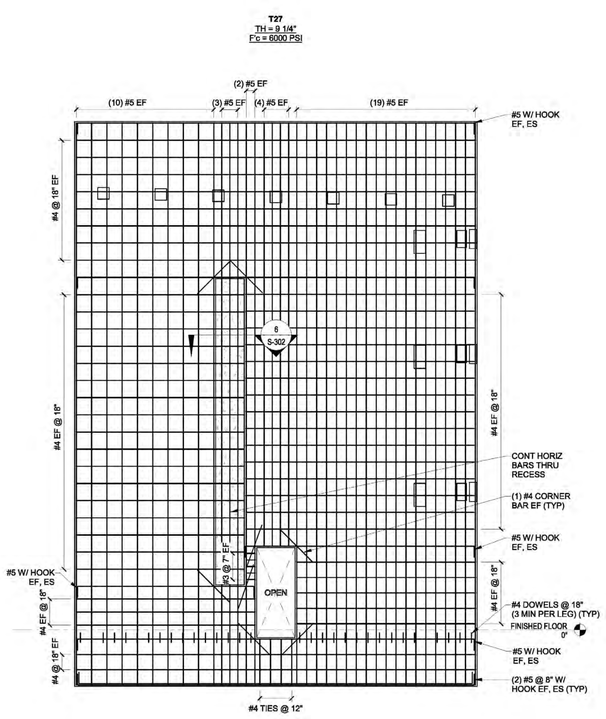
Date	Job No.
2024 JUNE 28	24.0000849.000

Sheet Title
TILT PANEL REBAR ELEVATIONS

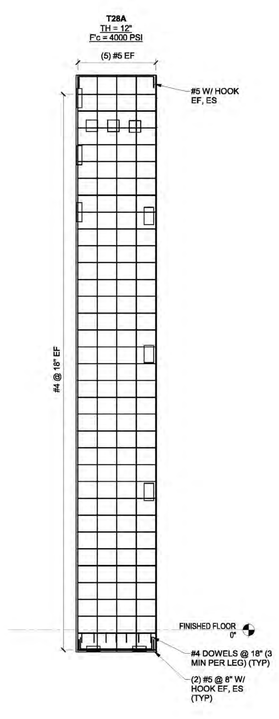
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CONSTRUCTION SET

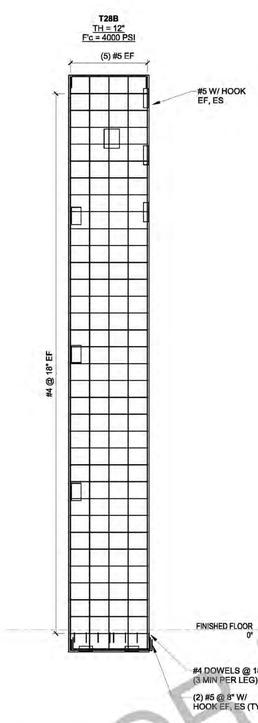
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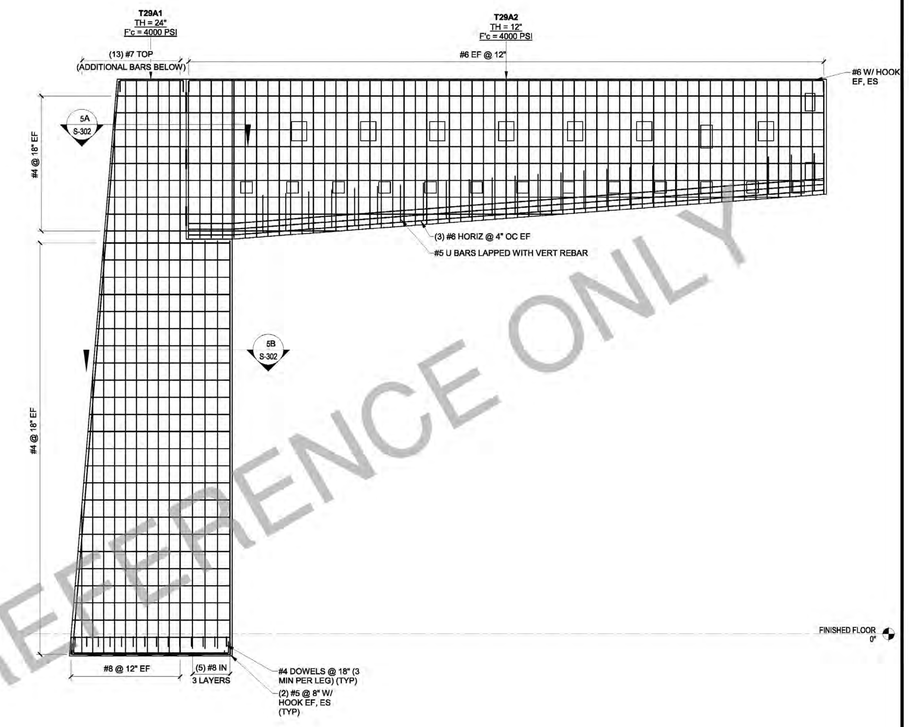
1 S-324 **T27 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



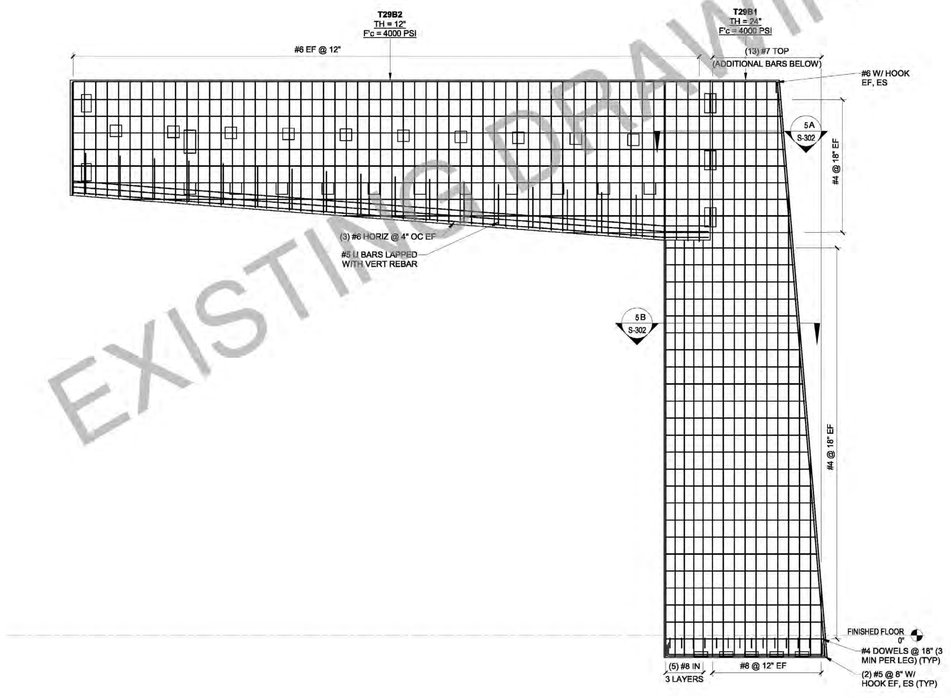
2 S-324 **T28A - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



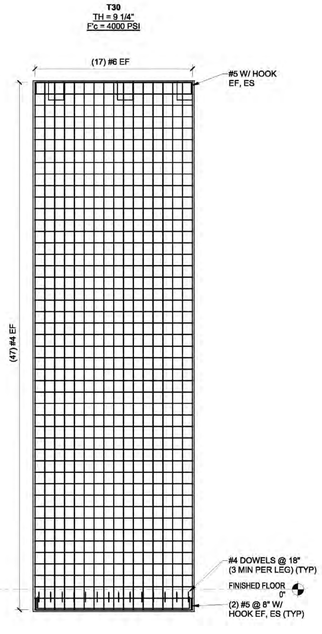
3 S-324 **T28B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



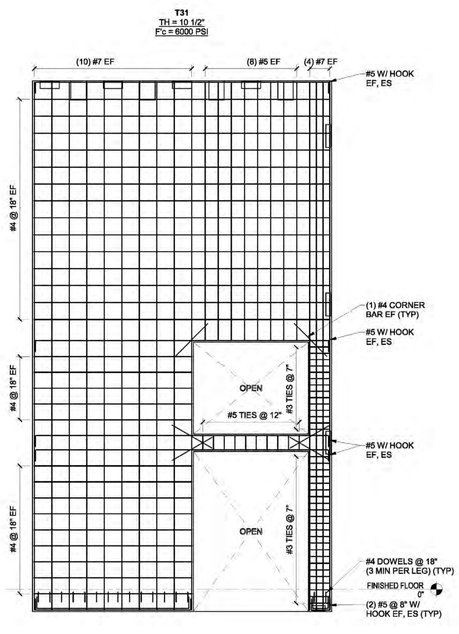
4 S-324 **T29A - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



5 S-324 **T29B - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



6 S-324 **T30 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-324 **T31 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
TILT PANEL REBAR ELEVATIONS

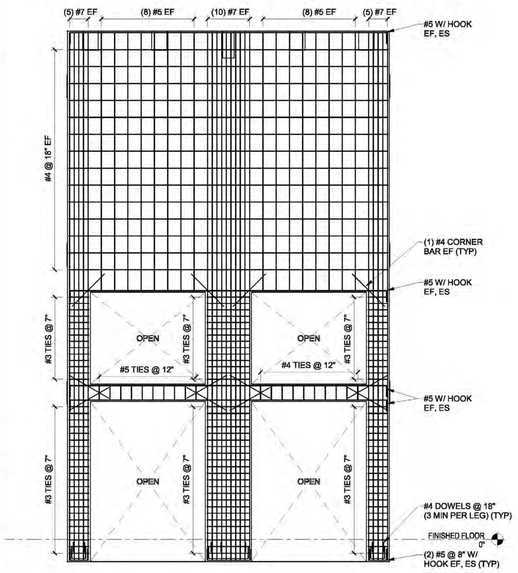
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S-324 (2)

CONSTRUCTION SET

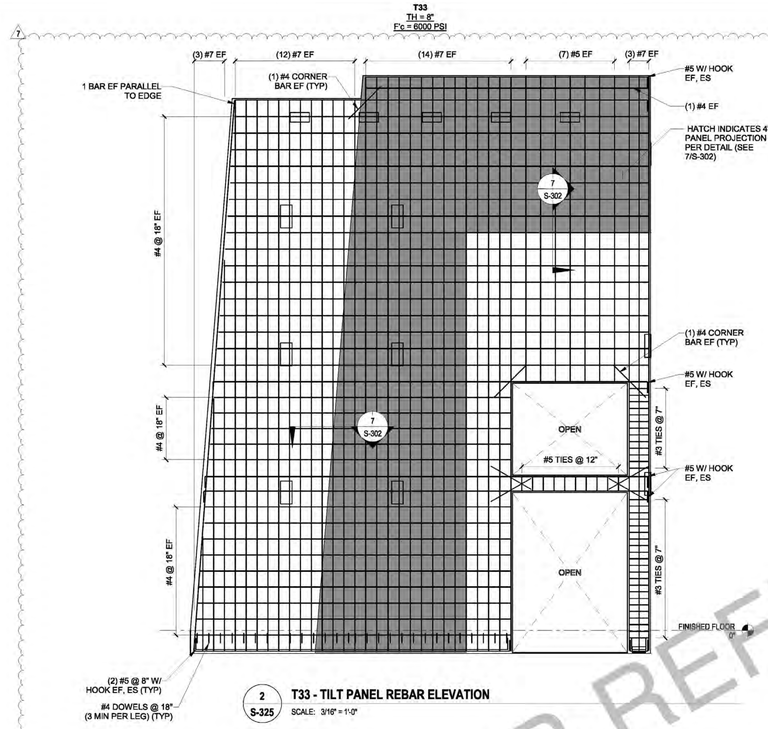
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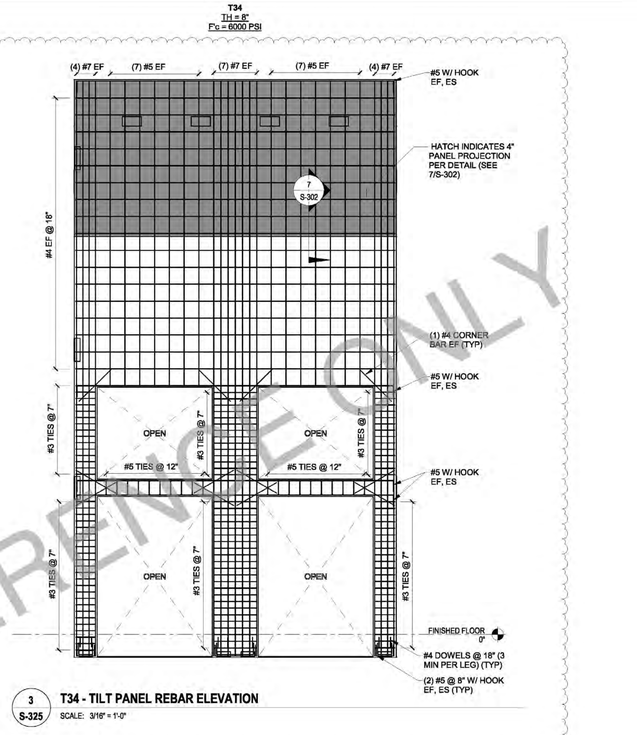
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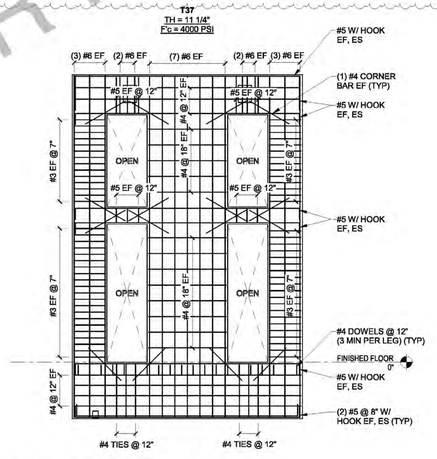
1 S-325 **T32 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



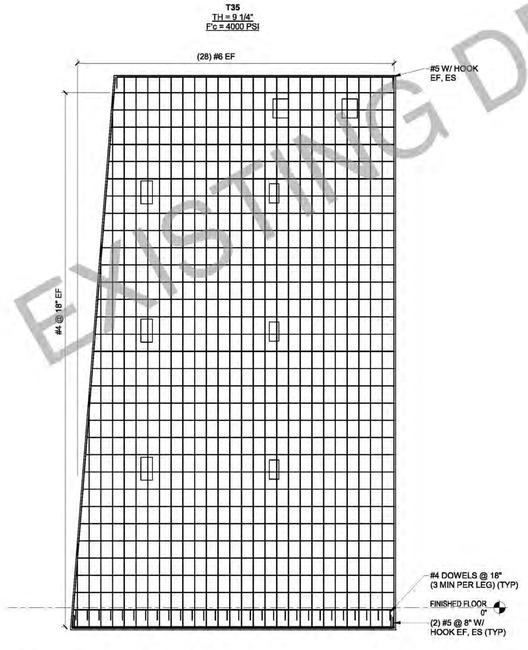
2 S-325 **T33 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



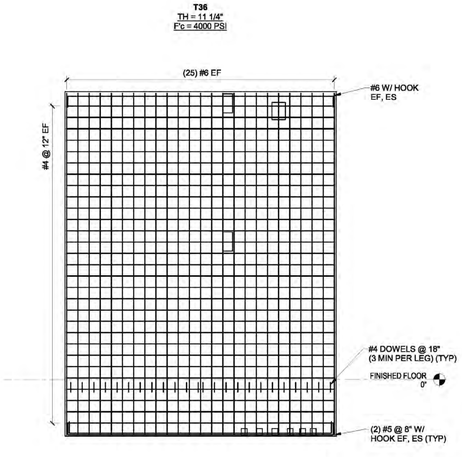
3 S-325 **T34 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



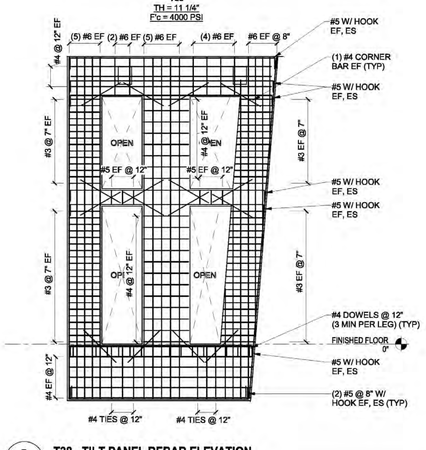
6 S-325 **T37 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



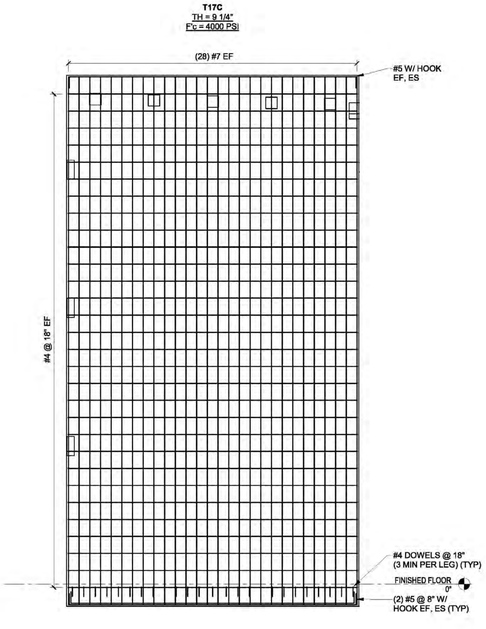
4 S-325 **T35 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



5 S-325 **T36 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



7 S-325 **T38 - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"



8 S-325 **T17C - TILT PANEL REBAR ELEVATION**
SCALE: 3/16" = 1'-0"

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Revisions

2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS
7	2025 FEBRUARY 28	RFI REPONSE

Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
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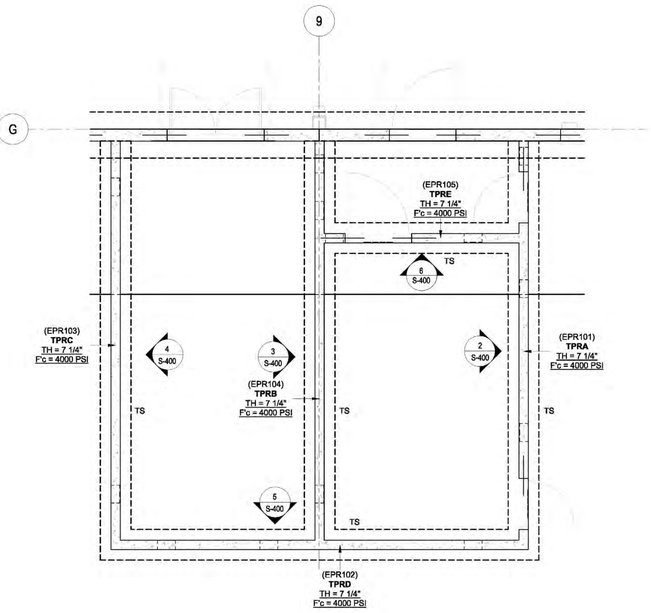
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S-325 2

CONSTRUCTION SET

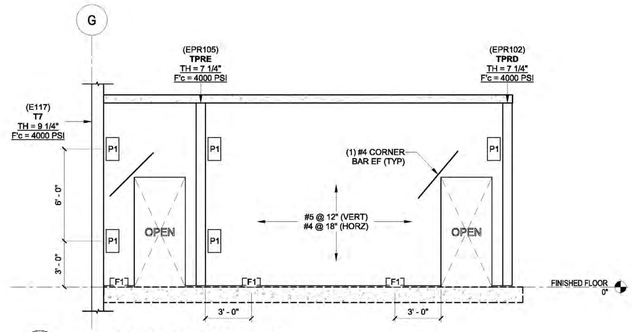
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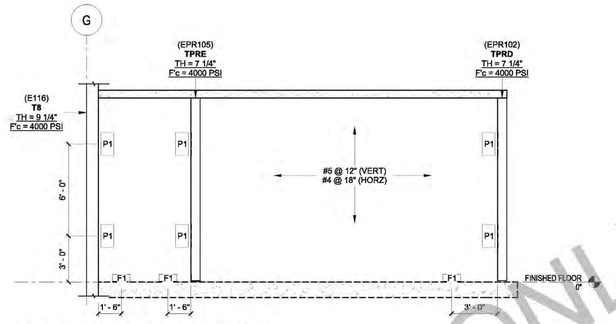
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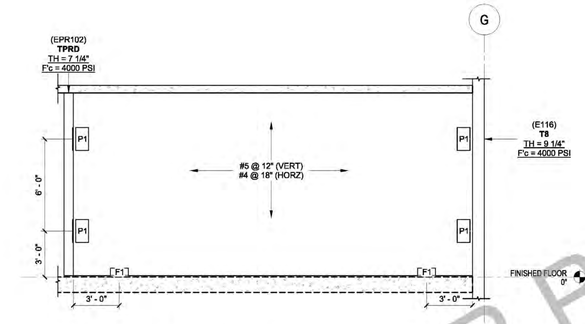
1 S-400 **PUMP ROOM FOUNDATION PLAN**
SCALE: 1/4" = 1'-0"



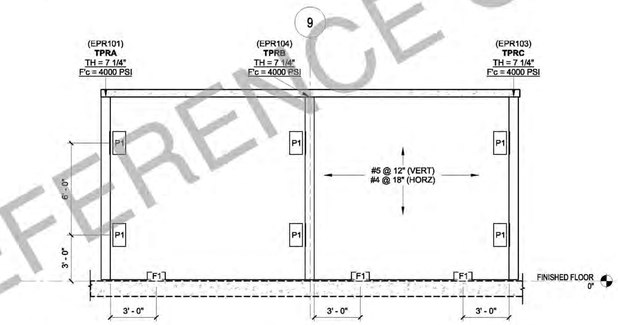
2 S-400 **TPRA - TILT PANEL REBAR ELEVATION**
SCALE: 1/4" = 1'-0"



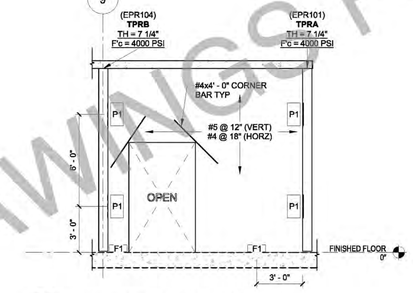
3 S-400 **TPRB - TILT PANEL REBAR ELEVATION**
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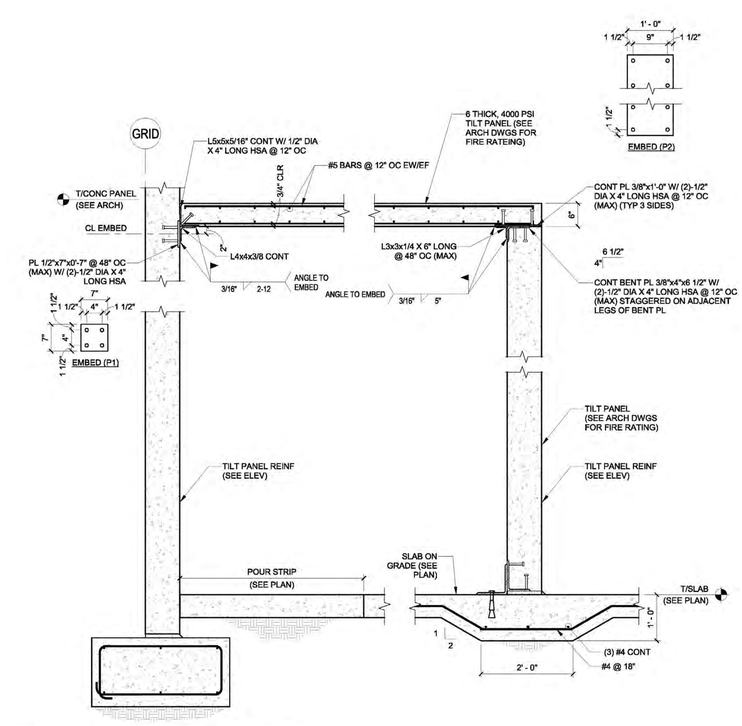
4 S-400 **TPRC - TILT PANEL REBAR ELEVATION**
SCALE: 1/4" = 1'-0"



5 S-400 **TPRD - TILT PANEL REBAR ELEVATION**
SCALE: 1/4" = 1'-0"



6 S-400 **TPRE - TILT PANEL REBAR ELEVATION**
SCALE: 1/4" = 1'-0"



7 S-400 **TYPICAL PUMP/ELECTRICAL ROOM SECTION**
SCALE: 3/4" = 1'-0"

TILT PANEL SCHEDULE

T1	TH = 9 1/4"	F'c = 4000 PSI
T2	TH = 9 1/4"	F'c = 4000 PSI
T3	TH = 9 1/4"	F'c = 4000 PSI
T4	TH = 9 1/4"	F'c = 4000 PSI
T5	TH = 9 1/4"	F'c = 4000 PSI
T6	TH = 9 1/4"	F'c = 4000 PSI
T6B	TH = 9 1/4"	F'c = 4000 PSI
T7	TH = 9 1/4"	F'c = 4000 PSI
T7B	TH = 9 1/4"	F'c = 4000 PSI
T8	TH = 9 1/4"	F'c = 4000 PSI
T9	TH = 9 1/4"	F'c = 4000 PSI
T10	TH = 9 1/4"	F'c = 6000 PSI
T11	TH = 9 1/4"	F'c = 4000 PSI
T12	TH = 9 1/4"	F'c = 4000 PSI
T13	TH = 9 1/4"	F'c = 4000 PSI
T14	TH = 9 1/4"	F'c = 4000 PSI
T15	TH = 9 1/4"	F'c = 4000 PSI
T16	TH = 9 1/4"	F'c = 4000 PSI
T17	TH = 9 1/4"	F'c = 4000 PSI
T17B	TH = 9 1/4"	F'c = 4000 PSI
T17C	TH = 9 1/4"	F'c = 4000 PSI
T18	TH = 9 1/4"	F'c = 4000 PSI
T19	TH = 11 1/4"	F'c = 6000 PSI
T19B	TH = 11 1/4"	F'c = 6000 PSI
T20	TH = 11 1/4"	F'c = 6000 PSI
T20B	TH = 11 1/4"	F'c = 6000 PSI
T21	TH = 11 1/4"	F'c = 6000 PSI
T21B	TH = 11 1/4"	F'c = 4000 PSI
T22	TH = 11 1/4"	F'c = 6000 PSI
T22B	TH = 11 1/4"	F'c = 4000 PSI
T23	TH = 9 1/4"	F'c = 4000 PSI
T23B	TH = 9 1/4"	F'c = 4000 PSI
T24	TH = 9 1/4"	F'c = 4000 PSI
T25	TH = 9 1/4"	F'c = 4000 PSI
T26	TH = 9 1/4"	F'c = 4000 PSI
T27	TH = 9 1/4"	F'c = 6000 PSI
T28A	TH = 12"	F'c = 4000 PSI
T28B	TH = 12"	F'c = 4000 PSI
T28A1	TH = 24"	F'c = 4000 PSI
T29A2	TH = 12"	F'c = 4000 PSI
T29B1	TH = 24"	F'c = 4000 PSI
T29B2	TH = 12"	F'c = 4000 PSI
T30	TH = 9 1/4"	F'c = 4000 PSI
T31	TH = 10 1/2"	F'c = 6000 PSI
T32	TH = 10 1/2"	F'c = 6000 PSI
T33	TH = 8"	F'c = 6000 PSI
T34	TH = 8"	F'c = 6000 PSI
T35	TH = 9 1/4"	F'c = 4000 PSI
T36	TH = 11 1/4"	F'c = 4000 PSI
T37	TH = 11 1/4"	F'c = 4000 PSI
T38	TH = 11 1/4"	F'c = 4000 PSI
TPRA	TH = 7 1/4"	F'c = 4000 PSI
TPRB	TH = 7 1/4"	F'c = 4000 PSI
TPRC	TH = 7 1/4"	F'c = 4000 PSI
TPRD	TH = 7 1/4"	F'c = 4000 PSI
TPRE	TH = 7 1/4"	F'c = 4000 PSI

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

Date 2024 JUNE 28
Job No. 24.0000849.000

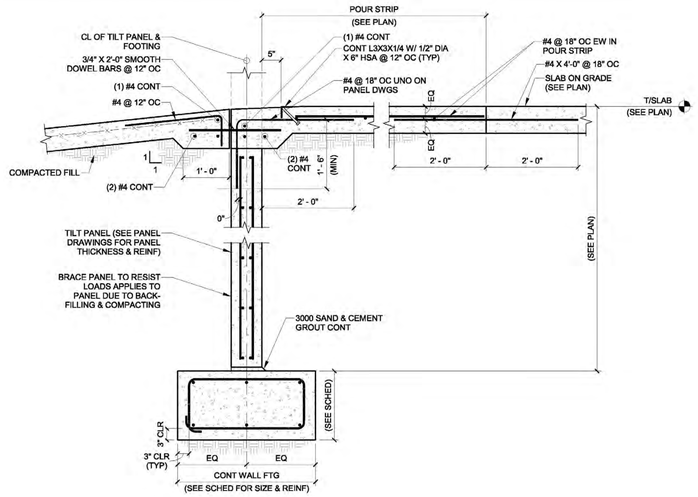
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PUMP ROOM PANEL PLAN & ELEVATIONS

Sheet No.
S-400

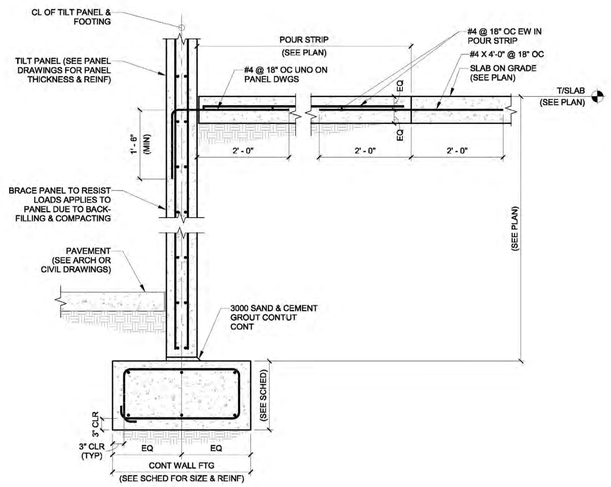
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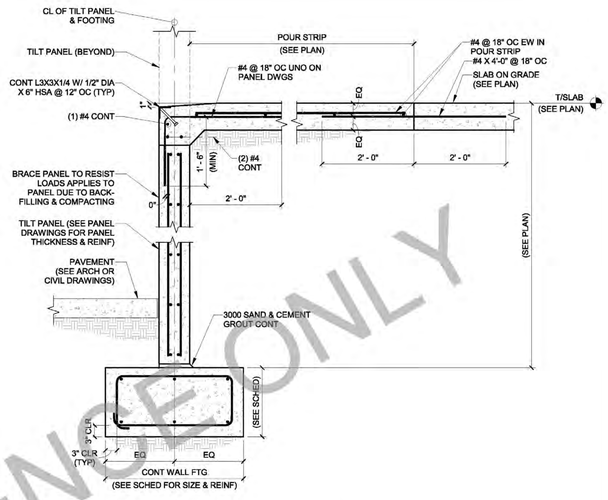
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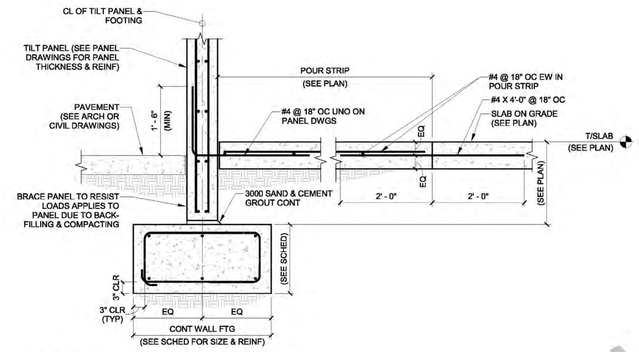
1 S-500 TYPICAL TILT PANEL FOOTING AT DRIVE-IN RAMP SECTION
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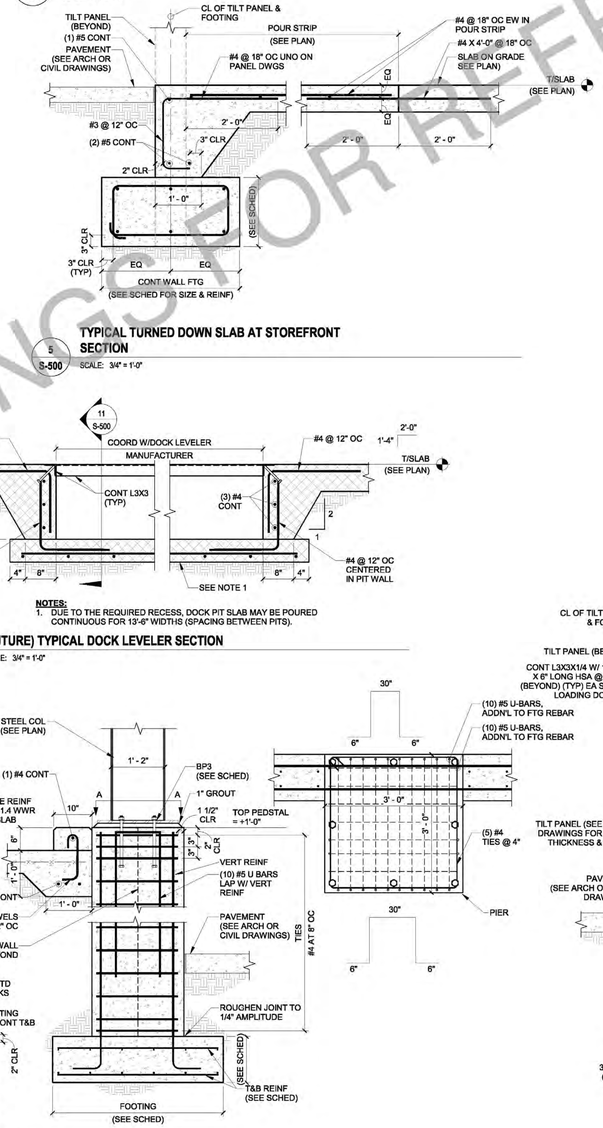
2 S-500 TILT PANEL FOOTING AT LOADING DOCK PANEL SECTION
SCALE: 3/4" = 1'-0"



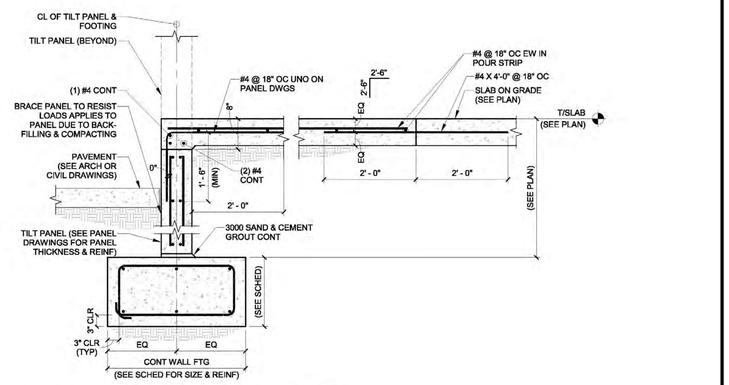
3 S-500 TYPICAL TILT PANEL FOOTING AT LOADING DOCK DOOR
SCALE: 3/4" = 1'-0"



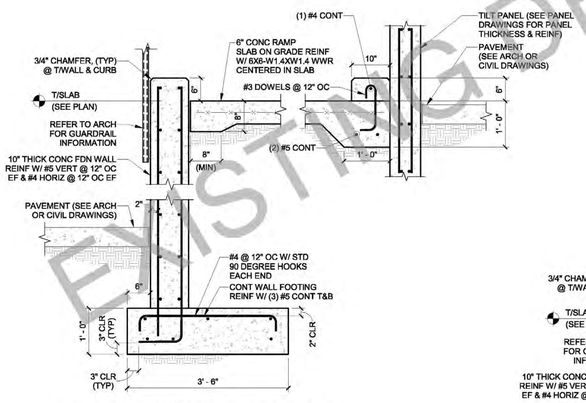
4 S-500 TYPICAL TILT PANEL FOOTING SECTION
SCALE: 3/4" = 1'-0"



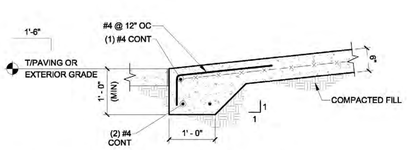
5 S-500 TYPICAL TURNED DOWN SLAB AT STOREFRONT SECTION
SCALE: 3/4" = 1'-0"



6 S-500 TYPICAL TILT PANEL FOOTING AT PERSONNEL DOOR SECTION
SCALE: 3/4" = 1'-0"



7 S-500 TYPICAL CONCRETE SLAB & FOUNDATION WALL AT DRIVE-IN RAMP SECTION
SCALE: 3/4" = 1'-0"

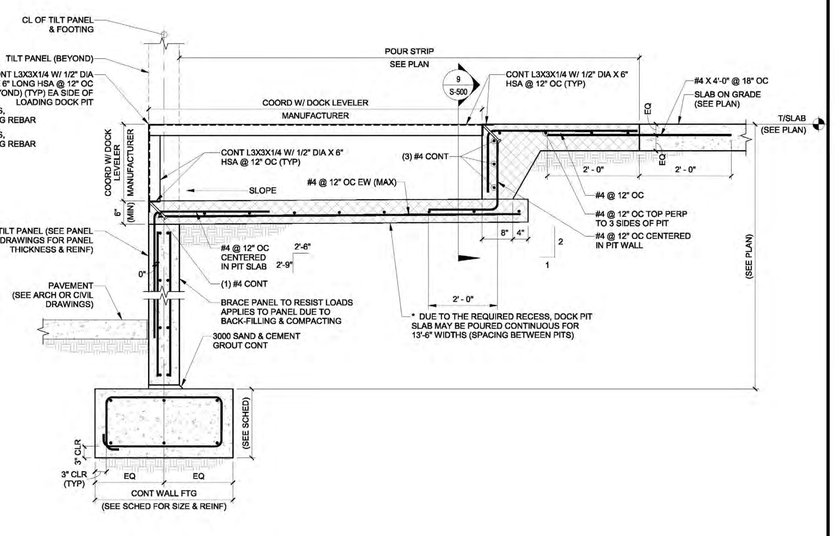


8 S-500 TYPICAL AT BOTTOM OF DRIVE-IN RAMP SECTION
SCALE: 3/4" = 1'-0"

10 S-500 TYPICAL CONCRETE SLAB & FOUNDATION WALL AT DRIVE-IN RAMP SECTION
SCALE: 3/4" = 1'-0"

NOTES:
1. DUE TO THE REQUIRED RECESS, DOCK PIT SLAB MAY BE POURED CONTINUOUS FOR 13'-6" WIDTHS (SPACING BETWEEN PITS).

9 S-500 (FUTURE) TYPICAL DOCK LEVELER SECTION
SCALE: 3/4" = 1'-0"



11 S-500 (FUTURE) TYPICAL DOCK LEVELER SECTION
SCALE: 3/4" = 1'-0"

EXISTING DRAWINGS FOR REFERENCE ONLY

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1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET
3	2024 OCTOBER 30	VE REVISIONS

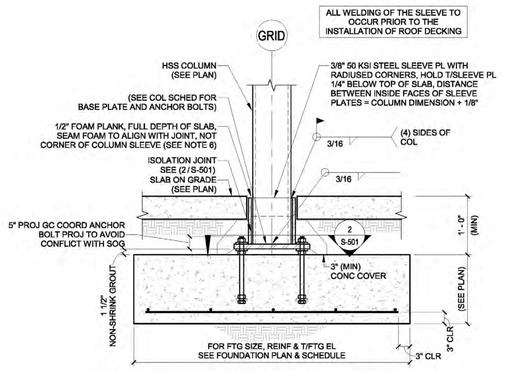
Date 2024 JUNE 28
Job No. 24.00008#9.000

Sheet Title
FOUNDATION SECTIONS & DETAILS

Sheet No.
S-500 2

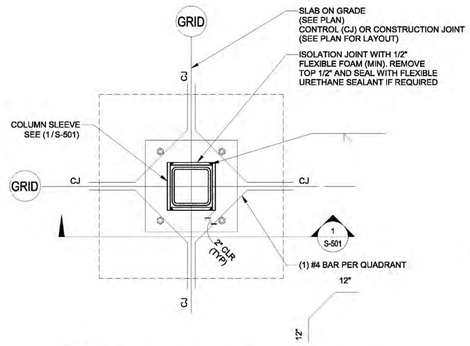
CONSTRUCTION SET

9.136



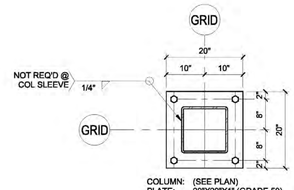
NOTES:
1. BOTTOM OF COLUMN MUST FULLY BEAR AGAINST BASE PLATE.
2. BASE PLATE AND COLUMN SLEEVE MUST BE PLUMBED PRIOR TO PLACING SLAB ON GRADE AND ERECTING COLUMN.
3. COLUMN SLEEVE MUST BE FREE FROM STANDING WATER PRIOR TO CAULKING.
4. PROVIDE 3 INCHES MINIMUM CONCRETE COVER FOR ALL STEEL.
5. IF CONCRETE COVER IS LESS THAN 3 INCHES, THEN APPLY (1) COAT BITUMOUS MASTIC COATING 10 MIL MINIMUM, COVER ALL EXPOSED STEEL BELOW SLAB.
6. ACCEPTABLE FLEXIBLE FOAM ISOLATION JOINT FILLER PRODUCTS:
   A. CERAMAR" BY WR MEADOWS.
   B. DECK-O-FOAM BY WR MEADOWS.
7. 5 INCHES MAXIMUM ANCHOR BOLT PROJECTION. GENERAL CONTRACTOR COORDINATE ANCHOR BOLT PROJECTION TO AVOID CONFLICT WITH CONCRETE SLAB.

1 / S-501 TYPICAL HSS COLUMN FOOTING SECTION
SCALE: 3/4" = 1'-0"



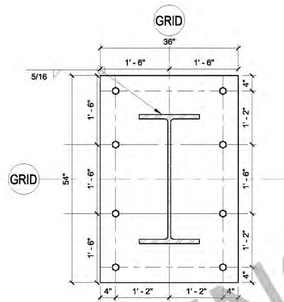
NOTES:
1. EXTEND SAW CUT TO EDGE OF FLEXIBLE FOAM INSULATION IMMEDIATELY AFTER PREVIOUS CUTS ARE MADE USING A SMALL DIAMETER DIAMOND BLADE.

2 / S-501 TYPICAL ISOLATION JOINT AT HSS COLUMN SECTION
SCALE: 3/4" = 1'-0"



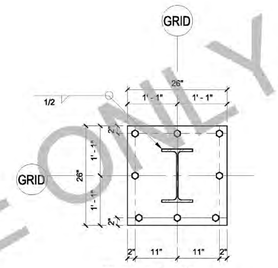
COLUMN: (SEE PLAN)
PLATE: 20"X20"X1" (GRADE 50)
BOLTS: (4) 3/4" DIA (GRADE 55) 12" EMBED

3 / S-501 BASE PLATE DETAIL - BP1
SCALE: 3/4" = 1'-0"



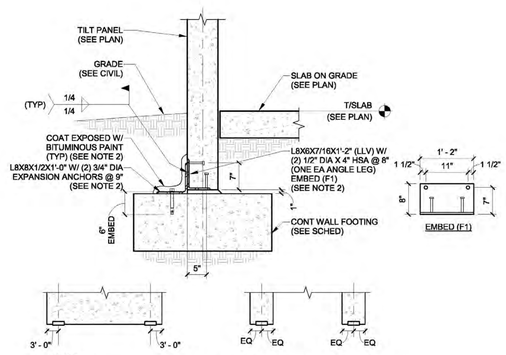
COLUMN: (SEE PLAN)
PLATE: 36"X54"X2" (GRADE 50)
BOLTS: (8) 2" DIA (F1554 GRADE 105) 20" EMBED

4 / S-501 BASE PLATE DETAIL - BP2
SCALE: 3/4" = 1'-0"



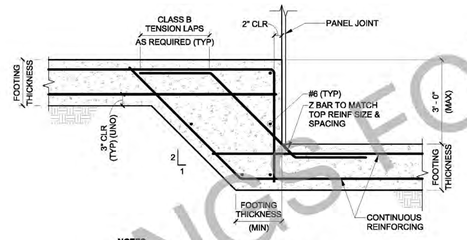
COLUMN: (SEE PLAN)
PLATE: 26"X26"X2 1/4" (GRADE 50)
BOLTS: (8) 1 1/2" DIA (F1554 GRADE 55) 18" EMBED

5 / S-501 BASE PLATE DETAIL - BP3
SCALE: 3/4" = 1'-0"



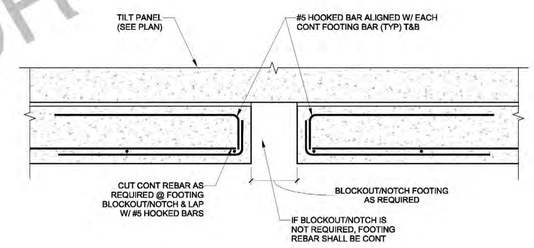
NOTES:
1. MINIMUM (2) EMBED CONNECTIONS PER PANEL WHERE REQUIRED.
2. EMBED NOT REQUIRED AT ALL PANEL LOCATIONS (SEE PANEL DRAWINGS FOR LOCATIONS OF CONNECTION).
3. SEE STRUCTURAL FOOTING SECTIONS AND DETAILS FOR ADDITIONAL INFORMATION.

6 / S-501 TYPICAL TILT PANEL TO FOOTING CONNECTION SECTION (F1)
SCALE: 3/4" = 1'-0"

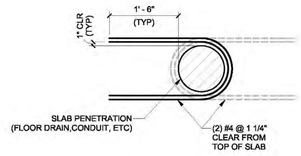


NOTES:
1. FOR FOOTING SIZE, REINFORCEMENT & ELEVATIONS (SEE FOUNDATION PLAN & CORRESPONDING FOOTING SECTION).
2. FOR FOOTING STEPS LOCATIONS (SEE FOUNDATION PLAN).
3. FOOTING TIES NOT SHOWN FOR CLARITY.

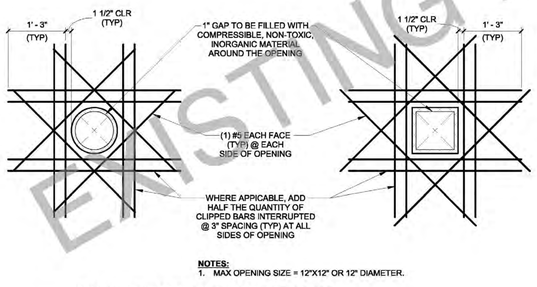
7 / S-501 TYPICAL FOOTING STEP SECTION
SCALE: 3/4" = 1'-0"



8 / S-501 TYPICAL NOTCH FOOTING DETAIL
SCALE: 3/4" = 1'-0"



9 / S-501 TYPICAL REINFORCEMENT AROUND OPENINGS DETAIL
SCALE: 3/4" = 1'-0"



NOTES:
1. MAX OPENING SIZE = 12"X12" OR 12" DIAMETER.

10 / S-501 TYPICAL REINFORCEMENT AROUND TILT PANEL OPENINGS DETAIL
SCALE: 3/4" = 1'-0"

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Date 2024 JUNE 28
Job No. 24.0000849.000

Sheet Title
FOUNDATION SECTIONS & DETAILS

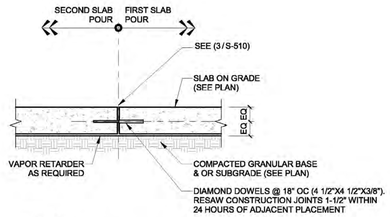
Sheet No.
S-501 2

CONSTRUCTION SET

9.137

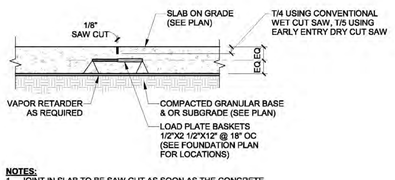
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PLOTTED: 2/26/2025 3:29:31 PM
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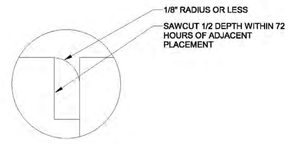
NOTES:
1. DOWELS TO BE AT RIGHT ANGLE TO JOINT.
2. ALL SURAFCES IN CONTACT WITH FUTURE PLACEMENTS SHALL BE CLEANED BEFORE PLACEMENT OF ADJACENT CONCRETE.

1 S-510 TYPICAL CONCRETE SLAB CONSTRUCTION JOINT DETAIL
SCALE: 3/4" = 1'-0"

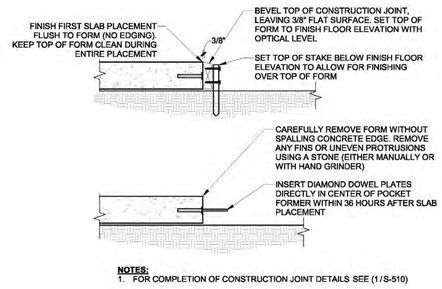


NOTES:
1. JOINT IN SLAB TO BE SAW CUT AS SOON AS THE CONCRETE WILL SAFELY SUPPORT WORKERS & EQUIPMENT AND WITHIN 4 HOURS OF PLACEMENT WITH EARLY ENTRY SAW OR 12 HOURS USING WET CUT SAW.
2. DOWELS TO BE AT RIGHT ANGLE TO JOINT.
3. LOAD PLATE BASKETS BY PNA.

2 S-510 TYPICAL CONCRETE SLAB CONTROL JOINT DETAIL
SCALE: 3/4" = 1'-0"

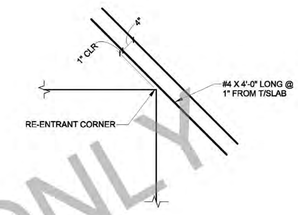


3 S-510 TYPICAL CONCRETE SLAB CONSTRUCTION JOINT DETAIL
SCALE: 3/4" = 1'-0"

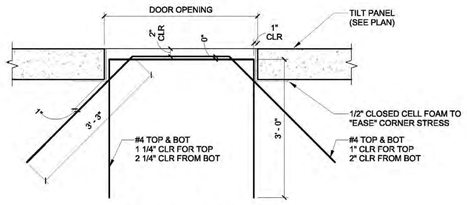


NOTES:
1. FOR COMPLETION OF CONSTRUCTION JOINT DETAILS SEE (1/S-510)

4 S-510 TYPICAL CONCRETE SLAB CONSTRUCTION JOINT INSTALLATION DETAIL
SCALE: 3/4" = 1'-0"

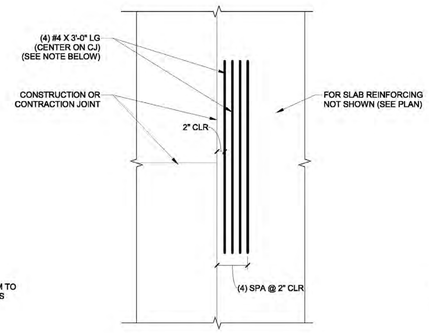


5 S-510 TYPICAL RE-ENTRANT REINFORCEMENT DETAIL
SCALE: 3/4" = 1'-0"



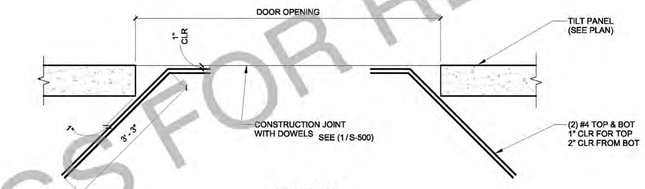
PERSONNEL DOOR

6 S-510 TYPICAL CONCRETE SLAB AT PERSONNEL DOOR DETAIL
SCALE: 3/4" = 1'-0"



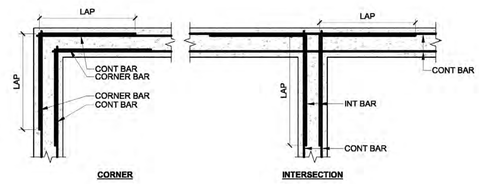
NOTES:
1. INSTALL REINFORCING 1 INCH CLEAR BELOW TOP OF SLAB.

7 S-510 TYPICAL DEAD END CONTROL JOINT REINFORCING DETAIL
SCALE: 3/4" = 1'-0"



RAMP DOOR

8 S-510 TYPICAL CONCRETE SLAB AT RAMP DOOR DETAIL
SCALE: 3/4" = 1'-0"



CORNER **INTERSECTION**

9 S-510 TYPICAL CONCRETE REINFORCING DETAIL
SCALE: NTS

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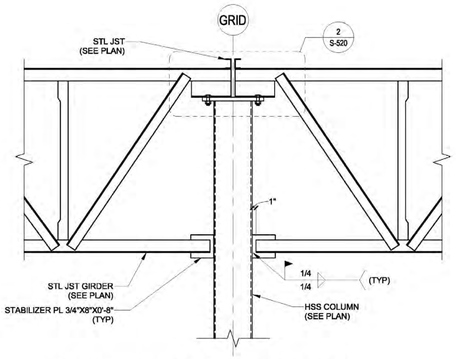
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Job No. 24.0000849.000

Sheet Title
SLAB ON GRADE SECTIONS & DETAILS

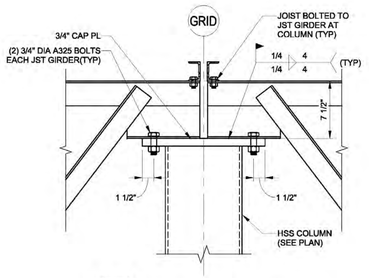
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S-510

CONSTRUCTION SET

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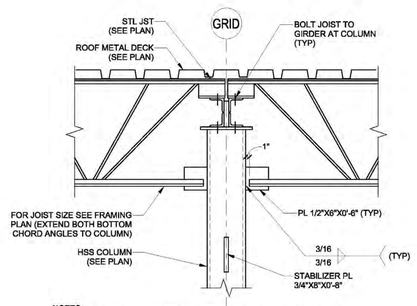


1 S-520 TYPICAL JOIST GIRDER TO HSS COLUMN CONNECTION SECTION
SCALE: 3/4" = 1'-0"



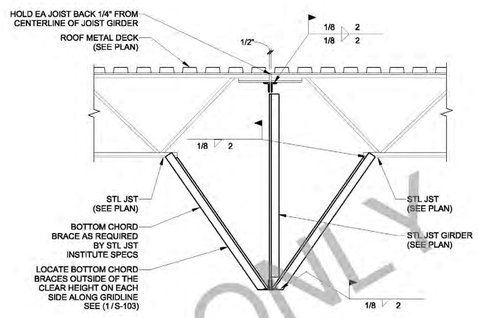
NOTES:
1. CAP PLATE 3/4" X COLUMN DIMENSION + 6" X COLUMN DIMENSION +3" CENTERED ON COLUMN.
2. DO NOT CONNECT JOIST OR JOIST GIRDER TO BOTTOM CHORD STABILIZER PLATE.

2 S-520 TYPICAL ENLARGED JOIST GIRDER CONNECTION TO HSS COLUMN SECTION
SCALE: 1 1/2" = 1'-0"

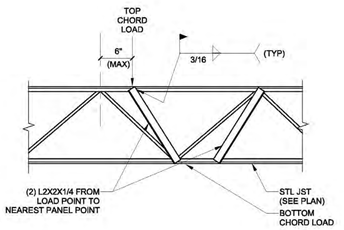


NOTES:
1. IF TOP PLATE IS WIDER THAN TOP CHORD OF JOIST, CONTACT ENGINEER OF RECORD FOR REVISED DETAIL.
2. GENERAL CONTRACTOR OPTION TO PROVIDE AN ALTERNATE CONNECTION DESIGN IN LIEU OF DETAIL SHOWN. CONTRACTOR TO PROVIDE SIGNED AND SEALED CALCULATIONS FOR EOR TO REVIEW. SEE SPECIAL JOIST DIAGRAMS ON S-103 FOR LOADS.

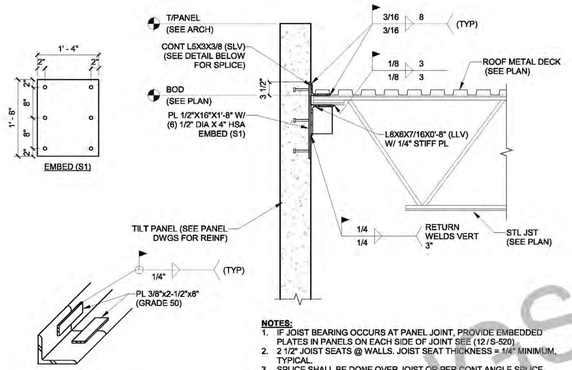
3 S-520 TYPICAL JOIST TO HSS COLUMN CONNECTION SECTION
SCALE: 3/4" = 1'-0"



4 S-520 TYPICAL JOIST ON JOIST GIRDER CONNECTION SECTION
SCALE: 3/4" = 1'-0"



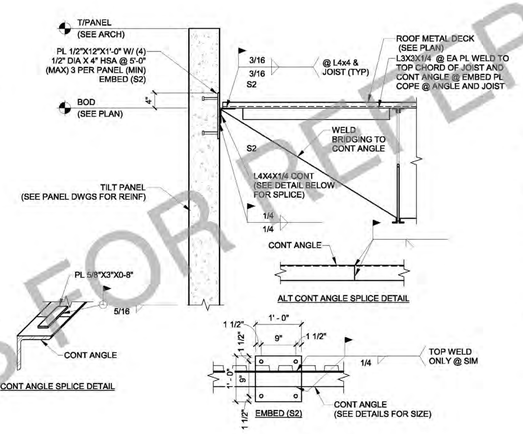
5 S-520 TYPICAL JOIST REINFORCING DETAIL
SCALE: NTS



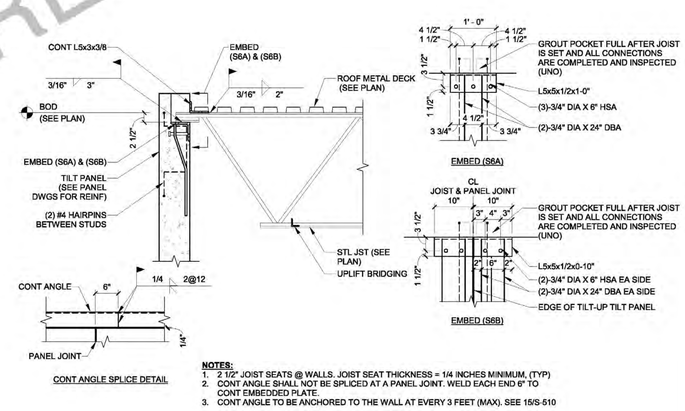
CONT ANGLE SPLICE DETAIL

NOTES:
1. IF JOIST BEARING OCCURS AT PANEL JOINT, PROVIDE EMBEDDED PLATES IN PANELS ON EACH SIDE OF JOINT SEE (12 / S-520)
2. 2 1/2" JOIST SEATS @ WALLS. JOIST SEAT THICKNESS = 1/4" MINIMUM, TYPICAL.
3. SPLICE SHALL BE DONE OVER JOIST OR PER CONT ANGLE SPLICE DETAIL.

6 S-520 TYPICAL JOIST BEARING ON TILT PANEL WITH PARAPET SECTION (S1)
SCALE: 3/4" = 1'-0"

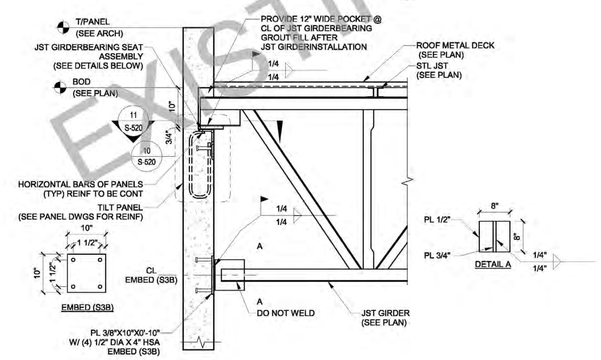


7 S-520 TYPICAL DECK BEARING ON TILT PANEL PARALLEL TO JOIST FRAMING SECTION (S2)
SCALE: 3/4" = 1'-0"

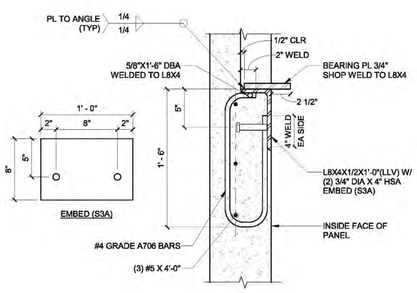


NOTES:
1. 2 1/2" JOIST SEATS @ WALLS. JOIST SEAT THICKNESS = 1/4 INCHES MINIMUM, (TYP)
2. CONT ANGLE SHALL NOT BE SPLICED AT A PANEL JOINT. WELD EACH END 6" TO CONT EMBEDDED PLATE.
3. CONT ANGLE TO BE ANCHORED TO THE WALL AT EVERY 3 FEET (MAX). SEE 15/S-510

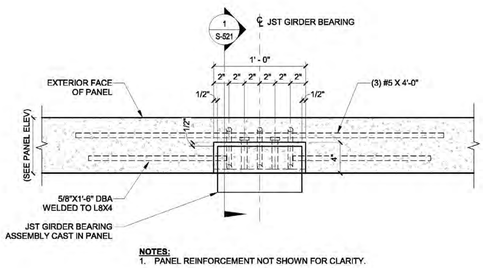
8 S-520 JOIST BEARING ON TILT PANEL (S6A) & (S6B)
SCALE: 3/4" = 1'-0"



9 S-520 TYPICAL JOIST GIRDER BEARING ON TILT PANEL SECTION (S3A) & (S3B)
SCALE: 3/4" = 1'-0"

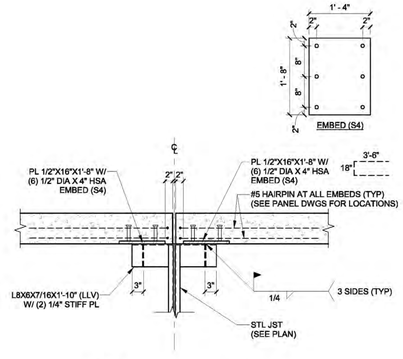


10 S-520 TYPICAL JOIST GIRDER BEARING ON TILT PANEL SECTION (S3A)
SCALE: 1 1/2" = 1'-0"



NOTES:
1. PANEL REINFORCEMENT NOT SHOWN FOR CLARITY.

11 S-520 TYPICAL JOIST GIRDER BEARING SEAT ASSEMBLY SECTION (S3A)
SCALE: 1 1/2" = 1'-0"



NOTES:
1. SEE PANEL ELEVATIONS FOR LOCATIONS.
2. FOR EMBED ELEVATION SEE (6 / S-520)

12 S-520 TYPICAL JOIST CONNECTION AT TILT PANEL JOINT SECTION (S4)
SCALE: 3/4" = 1'-0"

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Revisions

1	2024 MAY 25	PERMIT SET
2	2024 JUNE 28	CONSTRUCTION SET

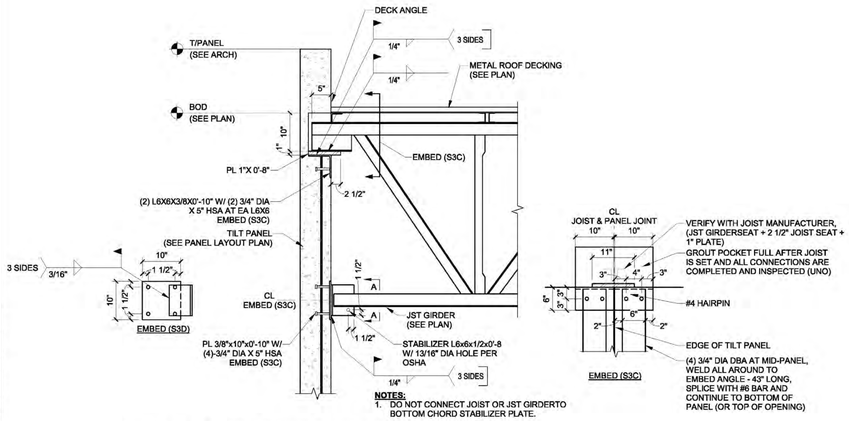
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Sheet Title
STEEL FRAMING SECTIONS & DETAILS

Sheet No.
S-520

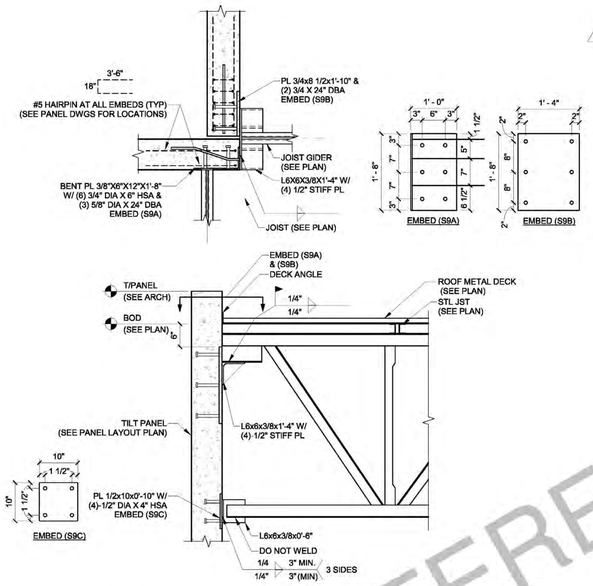
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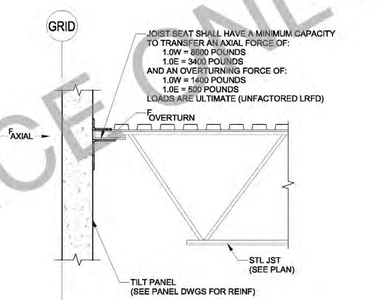


NOTES:
1. DO NOT CONNECT JOIST OR JST GIRDERTO BOTTOM CHORD STABILIZER PLATE.

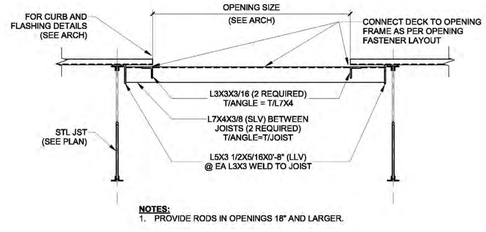
1 S-521 TYPICAL JOIST GIRDER BEARING AT ALL TILT PANEL JOINTS SECTION (S3C) & (S3D)
SCALE: 3/4" = 1'-0"



2 S-521 JOIST GIRDER CONNECTION (S9A) & (S9B) & (S9C)
SCALE: 3/4" = 1'-0"

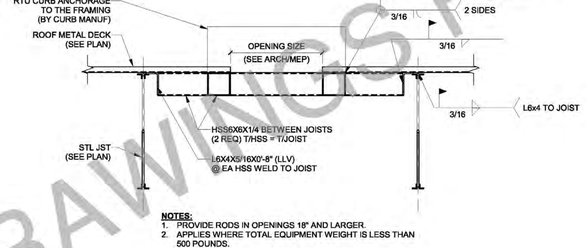


3 S-521 TYPICAL JOIST LOADING SECTION
SCALE: 3/4" = 1'-0"



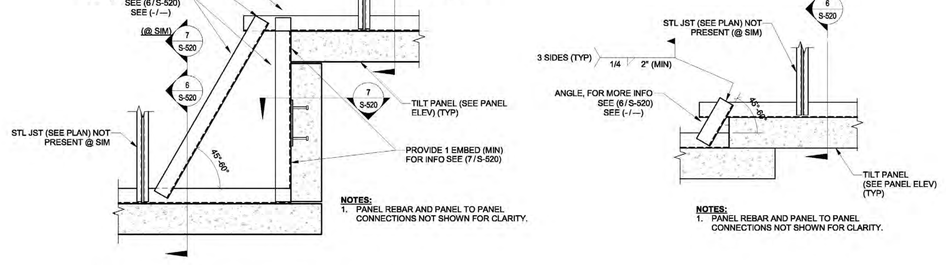
NOTES:
1. PROVIDE RODS IN OPENINGS 18" AND LARGER.

4 S-521 TYPICAL ROOF OPENING FRAMING SECTION
SCALE: 3/4" = 1'-0"



NOTES:
1. PROVIDE RODS IN OPENINGS 18" AND LARGER.
2. APPLIES WHERE TOTAL EQUIPMENT WEIGHT IS LESS THAN 500 POUNDS.

5 S-521 TYPICAL ROOF RTU FRAMING SUPPORT SECTION
SCALE: 3/4" = 1'-0"



NOTES:
1. PANEL REBAR AND PANEL TO PANEL CONNECTIONS NOT SHOWN FOR CLARITY.

6 S-521 TYPICAL CONNECTION AT OFFSET DECK EDGE ANGLES SECTION
SCALE: 3/4" = 1'-0"

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2	2024 JUNE 28	CONSTRUCTION SET

Date	Job No.
2024 JUNE 28	24.0000849.000

Sheet Title
STEEL FRAMING SECTIONS & DETAILS

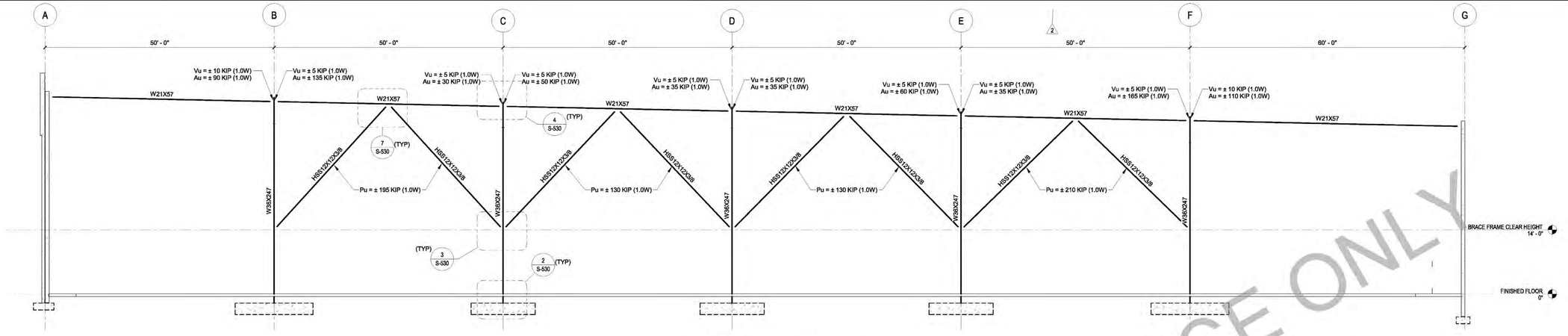
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S-521

CONSTRUCTION SET

9.140

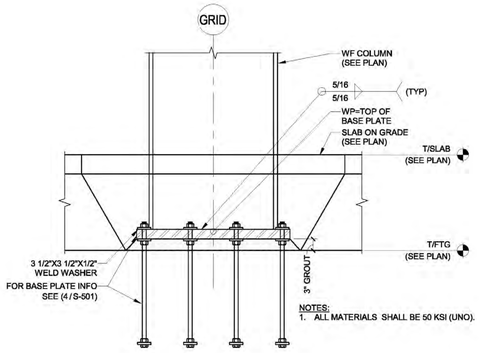
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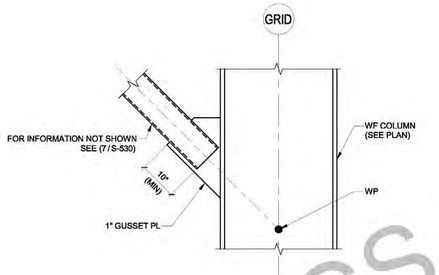


1 S-530 TYPICAL BRACED FRAME ELEVATION
SCALE: 1" = 10'-0"

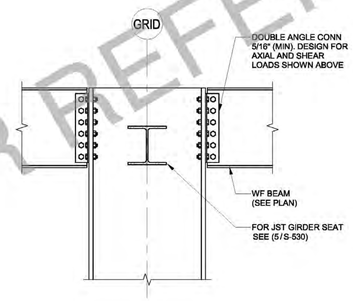
BRACED FRAME NOTES:
1. ALL BRACE FORCES ARE ULTIMATE (UNFACTORED LRFD) AND REVERSIBLE.
2. OMEGA FACTORS NOT INCLUDED WITH EQ LOADS.
3. (-) INDICATES FORCES ACTING UPWARD OR TO THE LEFT OR TENSION.
4. (+) INDICATES FORCES ACTING DOWNWARD OR TO THE RIGHT OR COMPRESSION.
5. BRACE FRAME CONNECTIONS ARE DELEGATED DESIGN. PROVIDE ENGINEER OF RECORD WITH CALCULATIONS AND DRAWING CERTIFIED BY A GEORGIA LICENSED PROFESSIONAL ENGINEER.



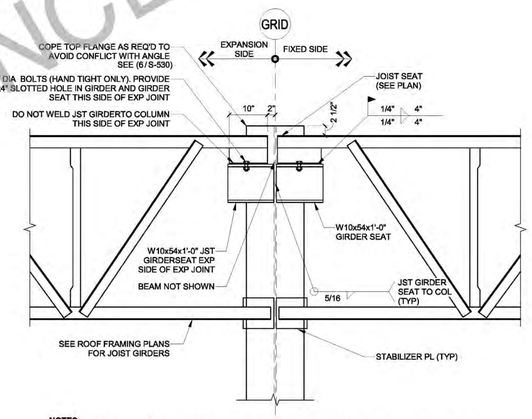
2 S-530 TYPICAL ANCHOR BOLT LAYOUT AT WF COLUMN SECTION
SCALE: 3/4" = 1'-0"



3 S-530 TYPICAL WF COLUMN TO HSS VERTICAL BRACING CONNECTION SECTION
SCALE: 3/4" = 1'-0"

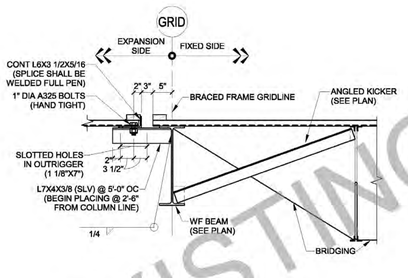


4 S-530 TYPICAL WF BEAM TO WF COLUMN CONNECTION SECTION
SCALE: 3/4" = 1'-0"

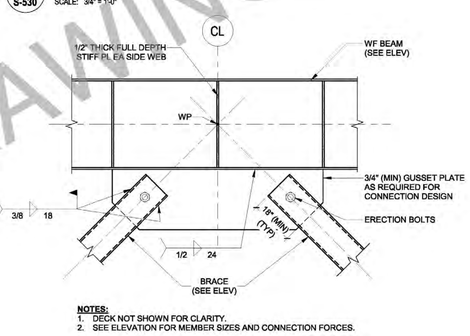


NOTES:
1. DO NOT WELD GIRDER ON EXPANSION JOINT SIDE TO COLUMN TOP PLATE OR STABILIZER PLATE BELOW.
2. FOR ADDITIONAL INFORMATION ON EXPANSION JOINT SEE (6/S-530)

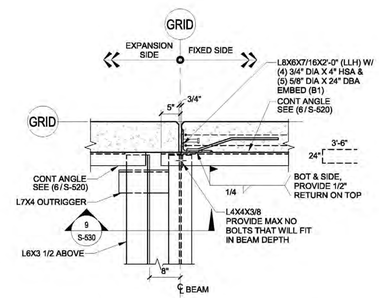
5 S-530 TYPICAL JOIST GIRDER BEARING AT WF COLUMN SECTION
SCALE: 3/4" = 1'-0"



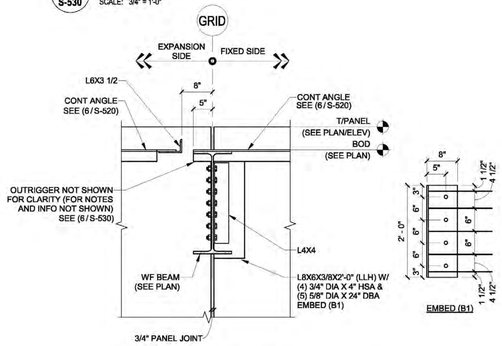
6 S-530 TYPICAL EXPANSION JOINT BRACED FRAME CONNECTION SECTION
SCALE: 3/4" = 1'-0"



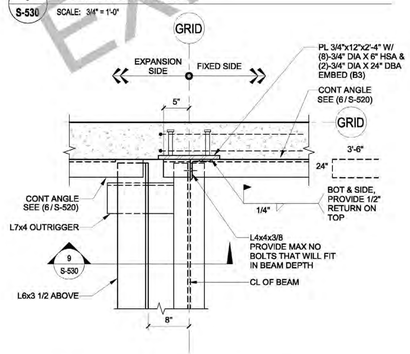
7 S-530 TYPICAL WF BEAM TO HSS VERTICAL BRAACING CONNECTION SECTION
SCALE: NTS



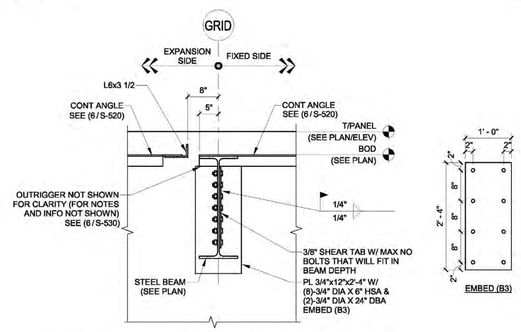
8 S-530 TYPICAL EXPANSION JOINT CONNECTION AT TILT PANEL WITH PARAPET (B1)
SCALE: 3/4" = 1'-0"



9 S-530 TYPICAL EXPANSION JOINT CONNECTION AT TILT PANEL WITH PARAPET (B1)
SCALE: 3/4" = 1'-0"



10 S-530 THRU EXPANSION JOINT WITHOUT A PANEL JOINT AT WALL WITH PARAPET (B3)
SCALE: 1" = 1'-0"



11 S-530 THRU EXPANSION JOINT WITHOUT A PANEL JOINT AT BEAM CONNECTION WITH PARAPET (B3)
SCALE: 3/4" = 1'-0"

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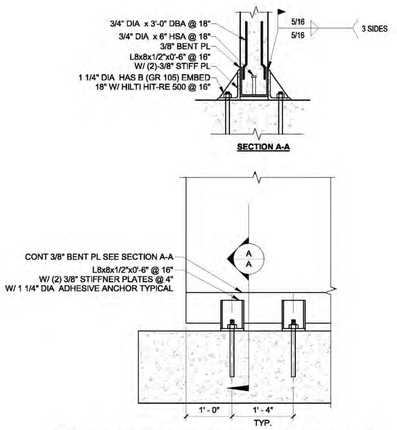
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BRACE FRAMING SECTIONS & DETAILS

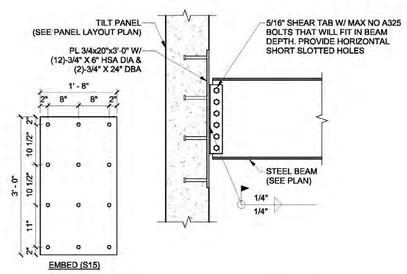
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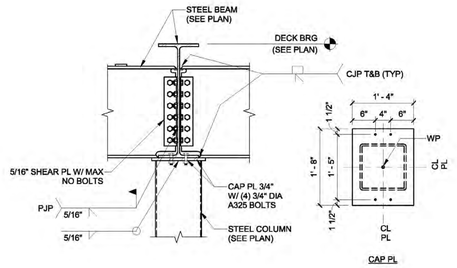
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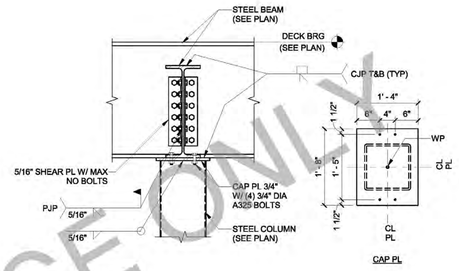
1 S-540 **TYPICAL TILT PANEL CONNECTION AT ENTRANCE (F2)**
SCALE: 3/4" = 1'-0"



2 S-540 **STEEL BEAM TO TILT PANEL CONNECTION (S15)**
SCALE: 3/4" = 1'-0"



3 S-540 **SECTION AT THRU BEAM TO COLUMN CONNECTION**
SCALE: 3/4" = 1'-0"



4 S-540 **SECTION AT THRU BEAM TO COLUMN CONNECTION**
SCALE: 3/4" = 1'-0"

5 S-540 **ENTRANCE CANOPY**
SCALE: 3/4" = 1'-0"

6 S-540 **DETAIL AT TYPICAL THRU BEAM MOMENT CONNECTION**
SCALE: 3/4" = 1'-0"

7 S-540 **WF BEAM TO WF BEAM CONNECTION SECTION**
SCALE: 3/4" = 1'-0"

8 S-540 **WF BEAM TO WF COLUMN MOMENT CONNECTION**
SCALE: 3/4" = 1'-0"

9 S-540 **WF BEAM TO WF COLUMN MOMENT CONNECTION SECTION**
SCALE: 3/4" = 1'-0"

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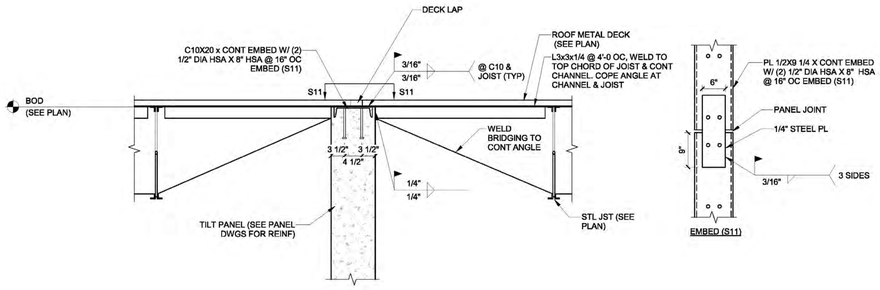
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ENTRANCE & PICKELBALL FRAMING SECTIONS & DETAILS

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S-540 /2

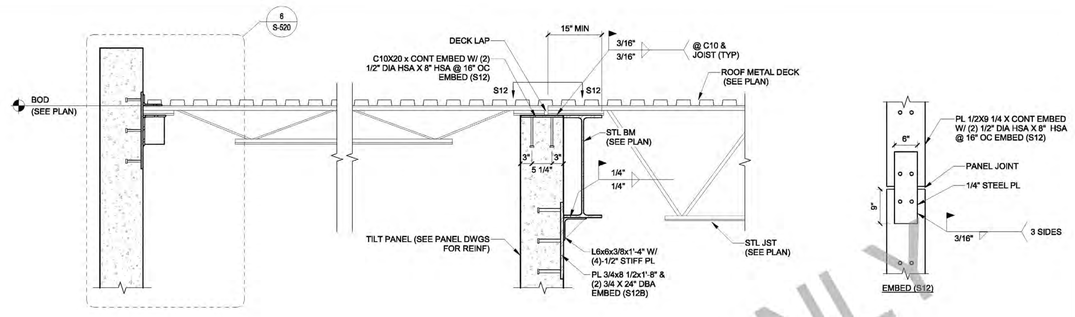
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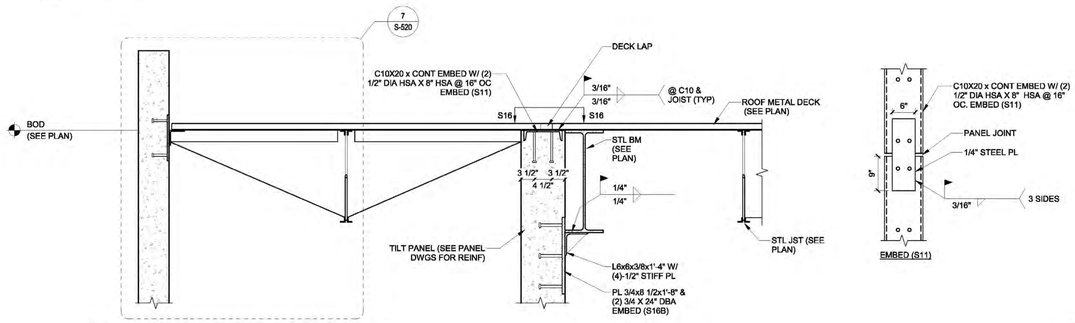
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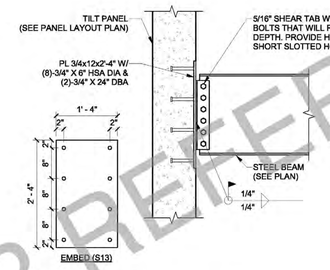
1 S-541 TYPICAL DECK BEARING ON WALL PARALLEL TO JOIST FRAMING (S11) & (S11B)
SCALE: 3/4" = 1'-0"



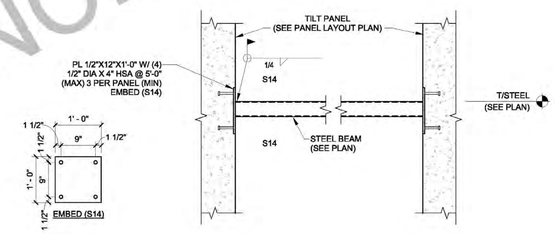
2 S-541 TYPICAL DECK BEARING ON WALL PARALLEL TO JOIST FRAMING (S12) & (S12B)
SCALE: 3/4" = 1'-0"



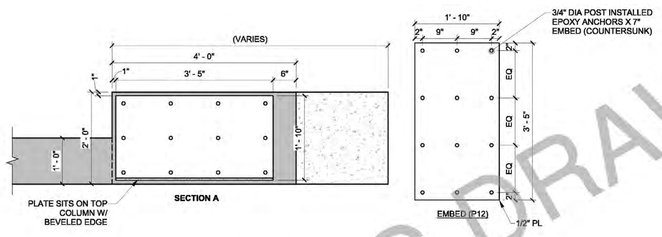
3 S-541 TYPICAL DECK BEARING ON WALL PARALLEL TO JOIST FRAMING (S16)
SCALE: 3/4" = 1'-0"

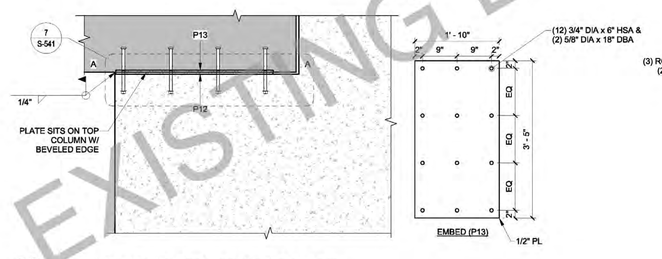


4 S-541 STEEL BEAM TO TILT PANEL CONNECTION AT PICKLEBALL (S13)
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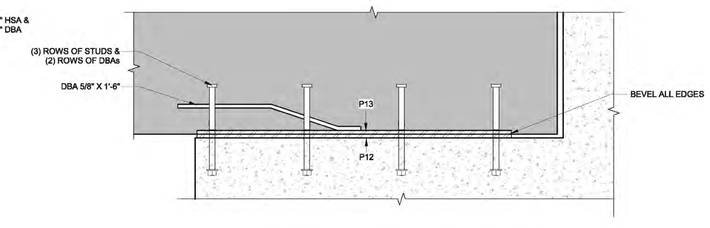


5 S-541 SOFFIT FRAMING EMBED CONNECTION (S14)
SCALE: 3/4" = 1'-0"





6 S-541 TILT PANEL BEARING ON EACHOTHER (P12) & (P13)
SCALE: 3/4" = 1'-0"



7 S-541 TILT PANEL BEARING ON EACHOTHER (P12) & (P13)
SCALE: 1 1/2" = 1'-0"

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2 2024 JUNE 28 CONSTRUCTION SET

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Sheet Title
ENTRANCE & PICKELBALL FRAMING SECTIONS & DETAILS

Sheet No.
S-541 2

CONSTRUCTION SET

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