Smith Monroe Gray
ENGINEERS, INC.

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MAPLE VALLEY
NEW LOADOUT ENCLOSURE

ENGINEER	STRUCTURAL CALCULATIONS	DATE
JWC	FOR	11/1/19

LAKESIDE INDUSTRIES

PROJECT NO.	REVISION
18-183F	1

CONTENTS:

11-1-19

JOSHUA W. CHRISTIANSEN
STATE OF WASHINGTON
51843
REGISTERED
PROFESSIONAL ENGINEER

DESIGN REFERENCES:

ASCE 7 MINIMUM DESIGN LOADS FOR BUILDING AND OTHER STRUCTURES
AISC STEEL CONSTRUCTION MANUAL
ACI 318 BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE

Project Description

Provide structural design for enclosure around load out and truck scale. Project is located 18825 SE Renton Maple Valley Road, Renton, WA 98058. SMG has completed other building and foundations at this site, for reference see drawing and calculations in SMG job number 18-183B.

Design Criteria

For more information on how these design values were calculated see 18-183B calcs Rev 1.

$RLL := 20\ psf$	Roof live load
$SL := 25\ psf$	Snow load
$V_u := 110\ mph$	Ultimate wind speed, Exposure C
$S_{DS} := 0.883$	Site specific seismic
$S_{D1} := 0.496$	
$R_1 := 3.25$	Ordinary Concetric braced frames
$R_2 := 3.5$	Ordinary Moment Frames

Geotechnical Parameters (per Geotech Consultants Inc. report dated 05/16/18)

$q_{allow} := 3000\ psf$	Allowable bearing pressure
$\mu_{soil} := 0.5$	Soil friction factor
$K_p := 350\ pcf$	Passive earth pressure

$$C_s := \frac{S_{DS}}{R_2} = 0.252$$

$$C_s := \frac{S_{DS}}{R_1} = 0.272$$

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E N G I N E E R S, I N C.

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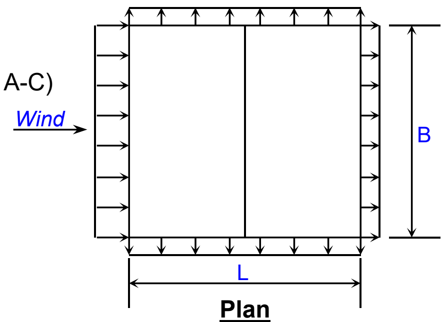
JOB NO. 18-183F SHEET 3 OF 32

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

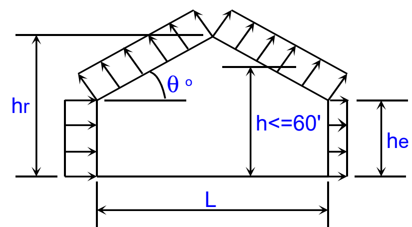
Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings
Using Chapter 28 Part 1: Envelope Procedure for Low-Rise Buildings

Input:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Risk Category =	II	(Table 1.5-1)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	19.83	ft. (hr >= he)
Eave Height, he =	18.00	ft. (he <= hr)
Building Width =	22.00	ft. (Normal to Building Ridge)
Building Length =	50.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	4.76	°	Slope: 7/8 in 12
Mean Roof Ht., h =	18.00	ft. (h = he, for angle <=10 deg.)	

Check Criteria for a Low-Rise Building:
1. Is h <= 60' ? Yes, Low-Rise   2. Is h <= Lesser of L or B? Yes, Low-Rise

External Pressure Coeff's., GCpf (Fig. 28.4-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 28.3-1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 28.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.88	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 28.3.2, Eq. 28.3-1)
qh = 23.23 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
Design Net External Wind Pressures (Sect. 28.4.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.4-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.4-1):

a =	3.00	ft.
2*a =	6.00	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.40	5.11	13.47	Zone 1	-0.45	-14.63	-6.27
Zone 2	-0.69	-20.21	-11.84	Zone 2	-0.69	-20.21	-11.84
Zone 3	-0.37	-12.77	-4.41	Zone 3	-0.37	-12.77	-4.41
Zone 4	-0.29	-10.92	-2.55	Zone 4	-0.45	-14.63	-6.27
Zone 5	---	---	---	Zone 5	0.40	5.11	13.47
Zone 6	---	---	---	Zone 6	-0.29	-10.92	-2.55
Zone 1E	0.61	9.99	18.35	Zone 1E	-0.48	-15.33	-6.97
Zone 2E	-1.07	-29.03	-20.67	Zone 2E	-1.07	-29.03	-20.67
Zone 3E	-0.53	-16.49	-8.13	Zone 3E	-0.53	-16.49	-8.13
Zone 4E	-0.43	-14.17	-5.81	Zone 4E	-0.48	-15.33	-6.97
Zone 5E	---	---	---	Zone 5E	0.61	9.99	18.35
Zone 6E	---	---	---	Zone 6E	-0.43	-14.17	-5.81

*Note: Use roof angle θ = 0 degrees for Longitudinal Direction.

For Case A when GCpf is neg. in Zones 2/2E: Zones 2/2E dist. = 11.00 ft.

For Case B when GCpf is neg. in Zones 2/2E: Zones 2/2E dist. = 25.00 ft.

Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	1.28	3.37	Zone 1T	---	-3.66	-1.57
Zone 2T	---	-5.05	-2.96	Zone 2T	---	-5.05	-2.96
Zone 3T	---	-3.19	-1.10	Zone 3T	---	-3.19	-1.10
Zone 4T	---	-2.73	-0.64	Zone 4T	---	-3.66	-1.57
Zone 5T	---	---	---	Zone 5T	---	1.28	3.37
Zone 6T	---	---	---	Zone 6T	---	-2.73	-0.64

Notes:
1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:
   - Zone 1 is windward wall for interior zone.
   - Zone 2 is windward roof for interior zone.
   - Zone 3 is leeward roof for interior zone.
   - Zone 4 is leeward wall for interior zone.
   - Zones 5 and 6 are sidewalls.
   - Zone 1T is windward wall for torsional case
   - Zone 3T is leeward roof for torsional case
   - Zones 5T and 6T are sidewalls for torsional case.
   - Zone 1E is windward wall for end zone.
   - Zone 2E is windward roof for end zone.
   - Zone 3E is leeward roof for end zone.
   - Zone 4E is leeward wall for end zone.
   - Zone 5E & 6E is sidewalls for end zone.
   - Zone 2T is windward roof for torsional case.
   - Zone 4T is leeward wall for torsional case.
2. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
3. Building must be designed for all wind directions using the 8 load cases shown below. The load cases are applied to each building corner in turn as the reference corner.
4. Wind loads for torsional cases are 25% of respective transverse or longitudinal zone load values. Torsional loading shall apply to all 8 basic load cases applied at each reference corner.
   Exception: One-story buildings with "h" <= 30', buildings <= 2 stories framed with light frame construction, and buildings <=2 stories designed with flexible diaphragms need not be designed for torsional load cases.
5. Per Code Section 28.4.4, the minimum wind load for MWFRS shall not be less than 16 psf. Both minimum load cases shall be applied as a separate load case in addition to the normal load cases determined in the above calculations.

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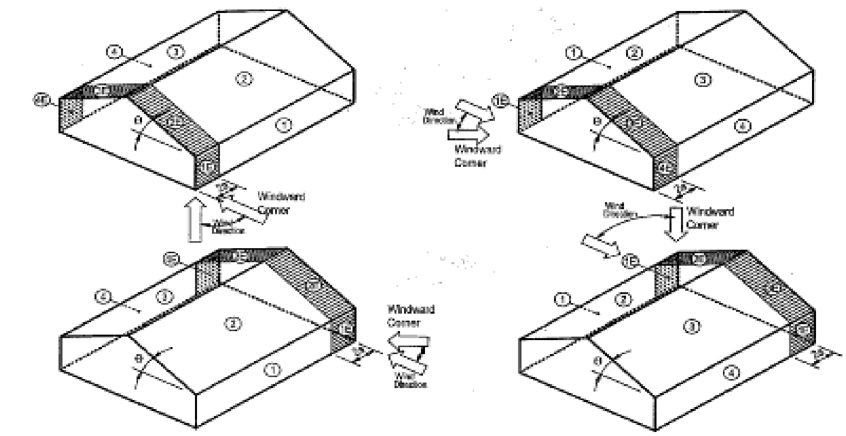
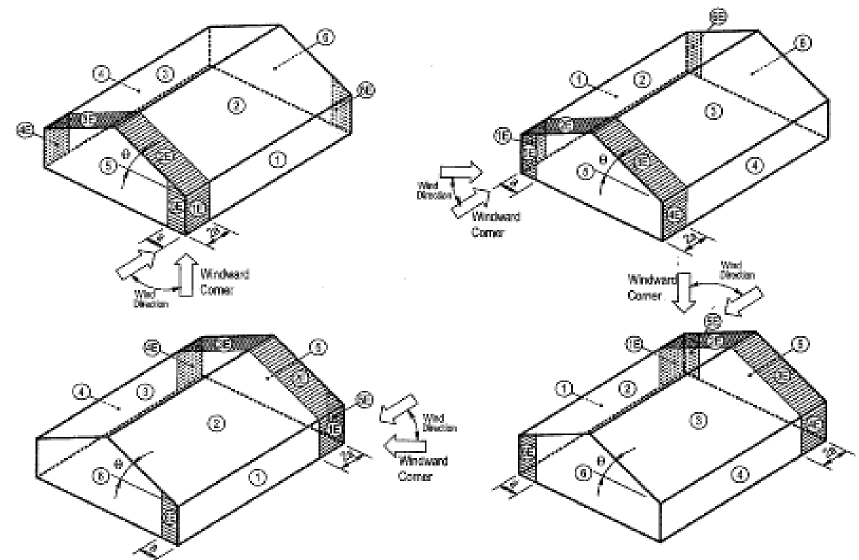
Low-Rise Buildings h <= 60'

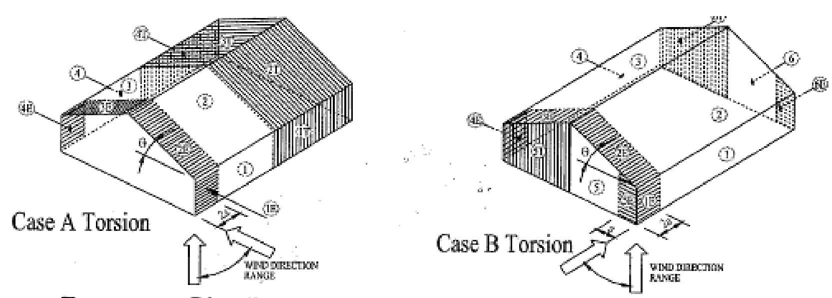


Load Case A



Load Case B

Basic Load Cases



Transverse Direction

Longitudinal Direction

Torsional Load Cases

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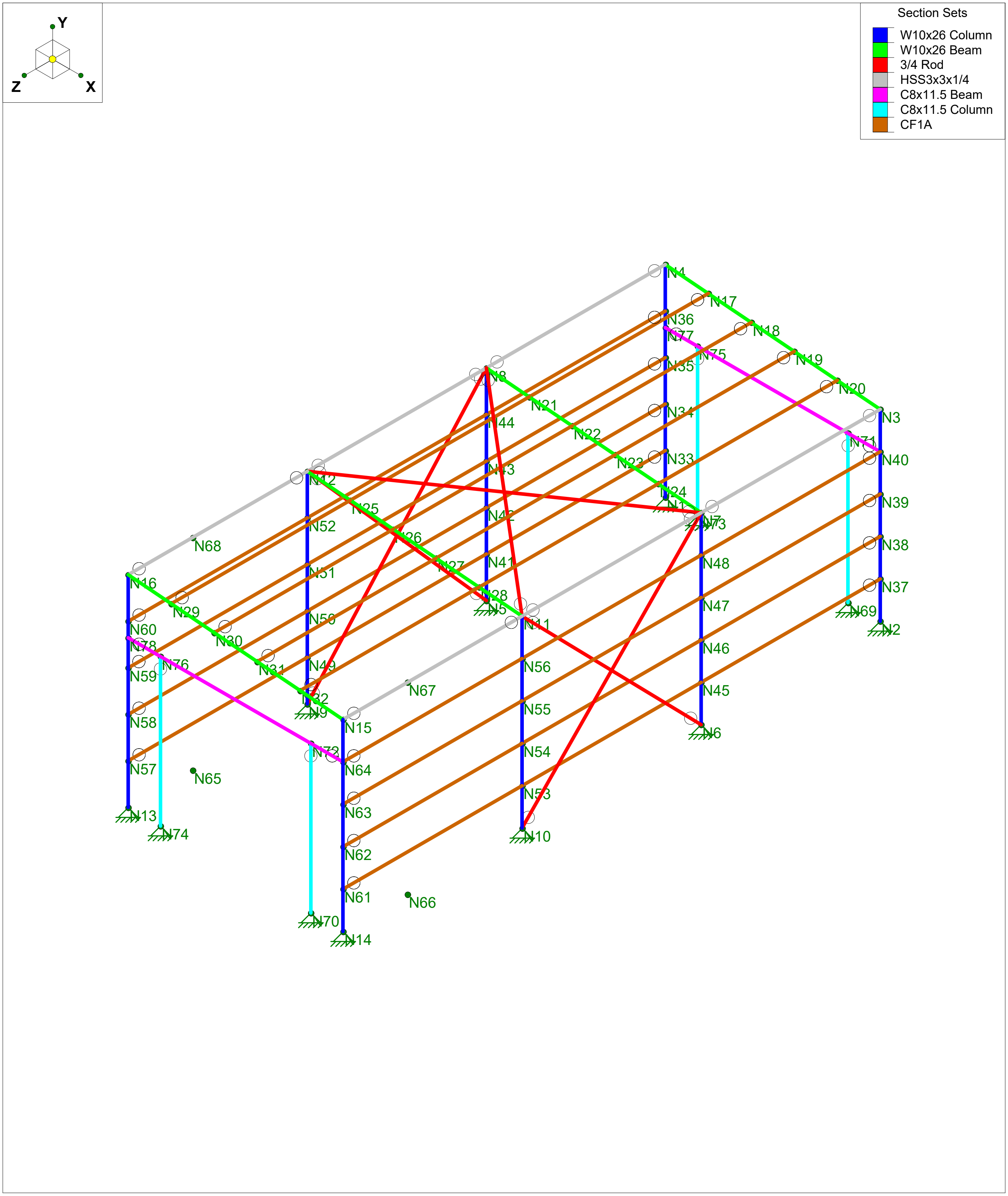


Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area(Me...	Surface(P...
1	Dead Load	DL		-1					9	
2	Snow Load	SL							1	
3	Wind Load X	WLX								
4	Wind Load Z	WLZ							6	
5	Earthquake Load X	ELX	-.25							
6	Earthquake Load Z	ELZ			-.27					
8	x1	WLX							6	
9	x2	OL1							6	
10	x3	OL2							6	
11	x4	OL3							6	
12		None						1		

Member Area Loads (BLC 1 : Dead Load)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N16	N15	N3	N4	Y	A-B	-.005
2	N14	N15	N3	N2	Y	A-B	-.005
3	N13	N16	N4	N1	Y	A-B	-.005
4	N78	N16	N15	N64	Y	A-B	-.005
5	N14	N70	N72	N64	Y	A-B	-.005
6	N74	N13	N78	N76	Y	A-B	-.005
7	N2	N69	N71	N40	Y	A-B	-.005
8	N7	N1	N77	N75	Y	A-B	-.005
9	N77	N4	N3	N40	Y	A-B	-.005

Member Area Loads (BLC 2 : Snow Load)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N16	N15	N3	N4	Y	A-B	-.025

Member Area Loads (BLC 4 : Wind Load Z)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N13	N74	N76	N78	Z	A-B	-.02
2	N70	N14	N64	N72	Z	A-B	-.02
3	N78	N16	N15	N64	Z	A-B	-.02
4	N77	N75	N73	N1	Z	A-B	-.02
5	N4	N77	N40	N3	Z	A-B	-.02
6	N71	N69	N2	N40	Z	A-B	-.02

Member Area Loads (BLC 8 : x1)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N14	N15	N67	N66	X	A-B	-.01
2	N15	N16	N68	N67	Perp	A-B	-.029
3	N13	N16	N68	N65	X	A-B	.015
4	N3	N2	N66	N67	X	A-B	-.005
5	N68	N67	N3	N4	Perp	A-B	.02
6	N65	N68	N4	N1	X	A-B	.011

Member Area Loads (BLC 9 : x2)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N14	N15	N67	N66	X	A-B	-.018
2	N15	N16	N68	N67	Perp	A-B	-.02
3	N13	N16	N68	N65	X	A-B	.006

Member Area Loads (BLC 9 : x2) (Continued)

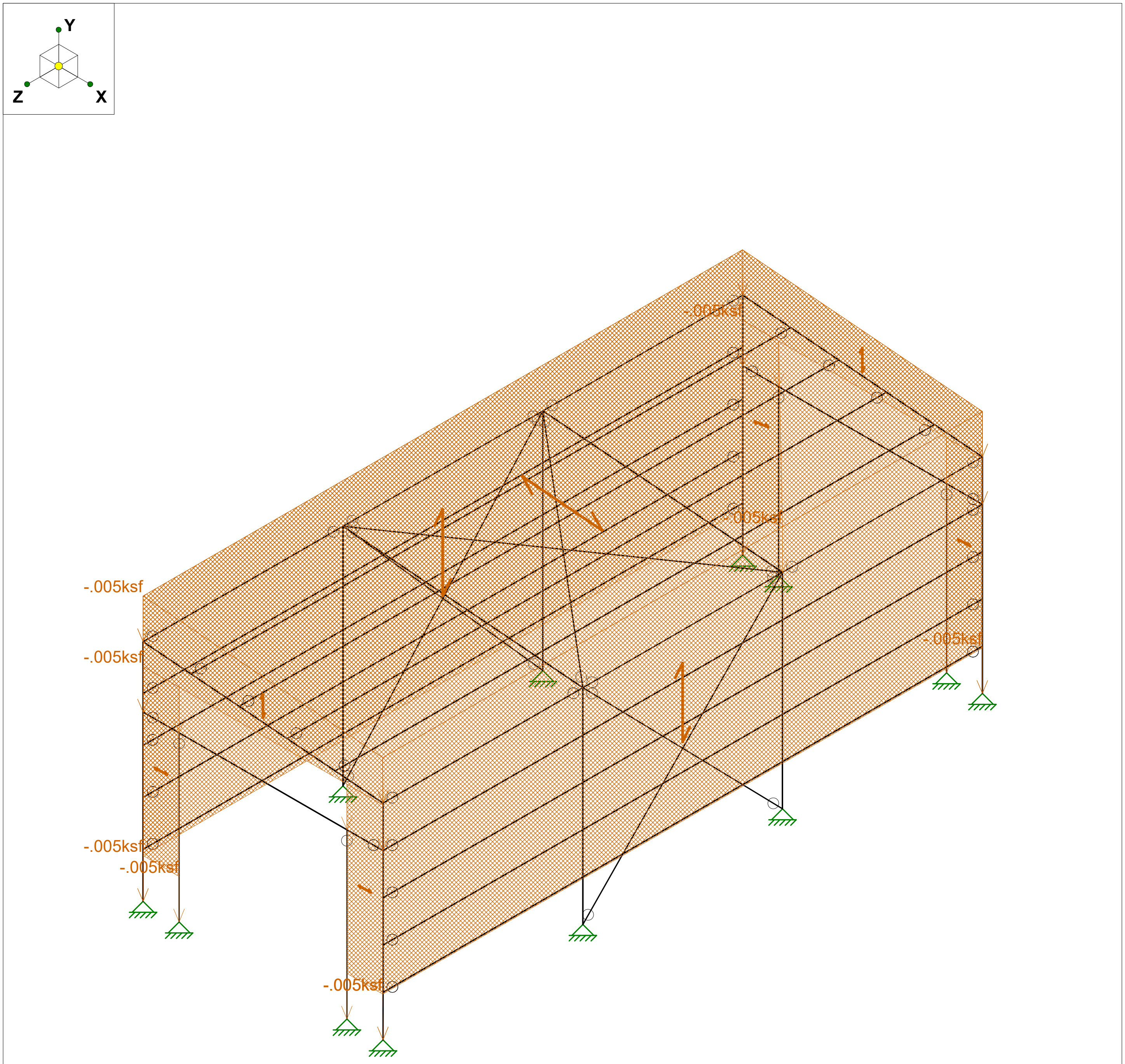
	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
4	N3	N2	N66	N67	X	A-B	-.013
5	N68	N67	N3	N4	Perp	A-B	.012
6	N65	N68	N4	N1	X	A-B	-.003

Member Area Loads (BLC 10 : x3)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N14	N15	N67	N66	X	A-B	.015
2	N15	N16	N68	N67	Perp	A-B	-.029
3	N13	N16	N68	N65	X	A-B	-.015
4	N3	N2	N66	N67	X	A-B	.015
5	N68	N67	N3	N4	Perp	A-B	.02
6	N65	N68	N4	N1	X	A-B	-.015

Member Area Loads (BLC 11 : x4)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N14	N15	N67	N66	X	A-B	.007
2	N15	N16	N68	N67	Perp	A-B	-.021
3	N13	N16	N68	N65	X	A-B	-.007
4	N3	N2	N66	N67	X	A-B	.006
5	N68	N67	N3	N4	Perp	A-B	.012
6	N65	N68	N4	N1	X	A-B	-.006



Loads: BLC 1, Dead Load

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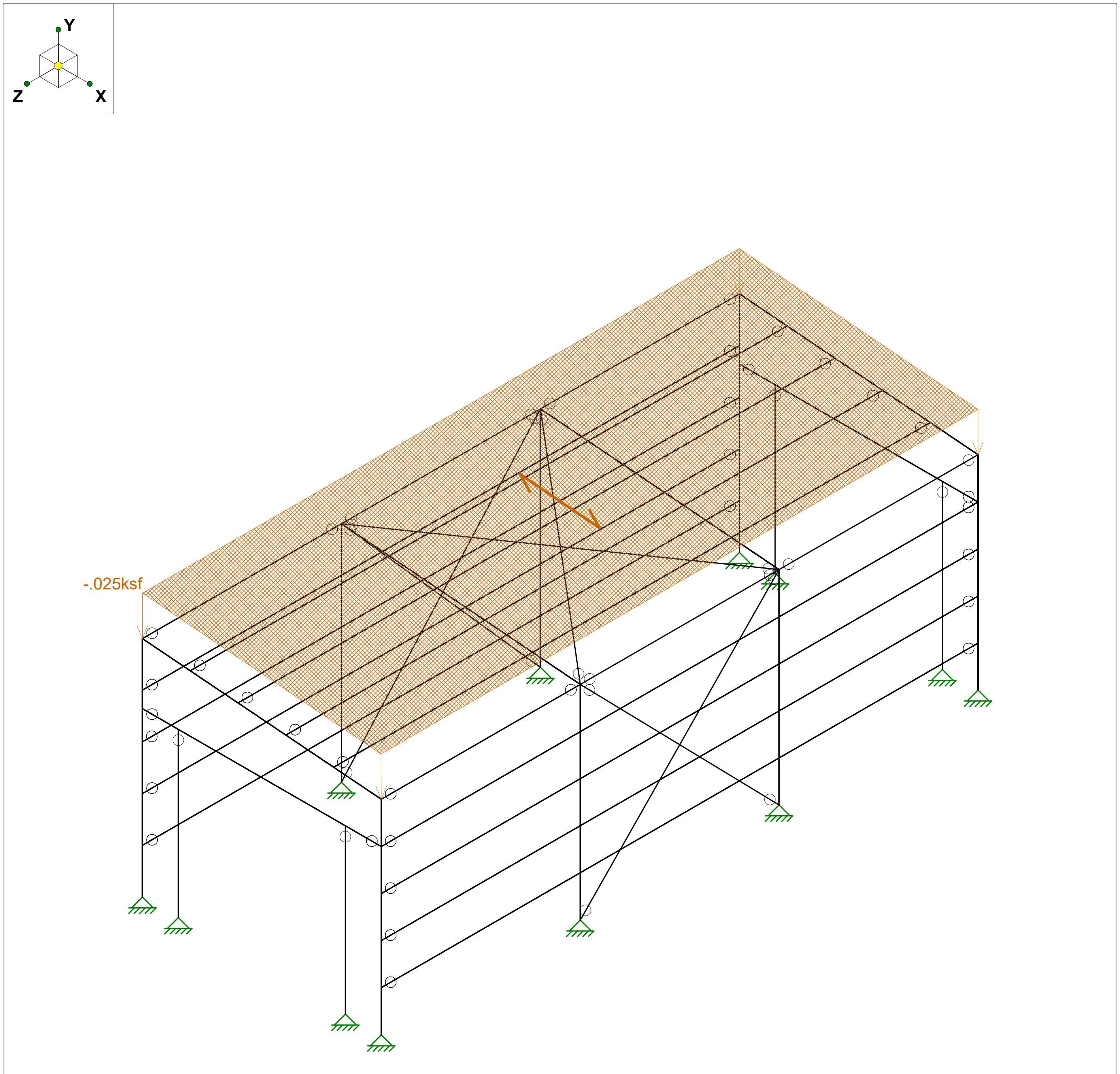

Loads: BLC 2, Snow Load

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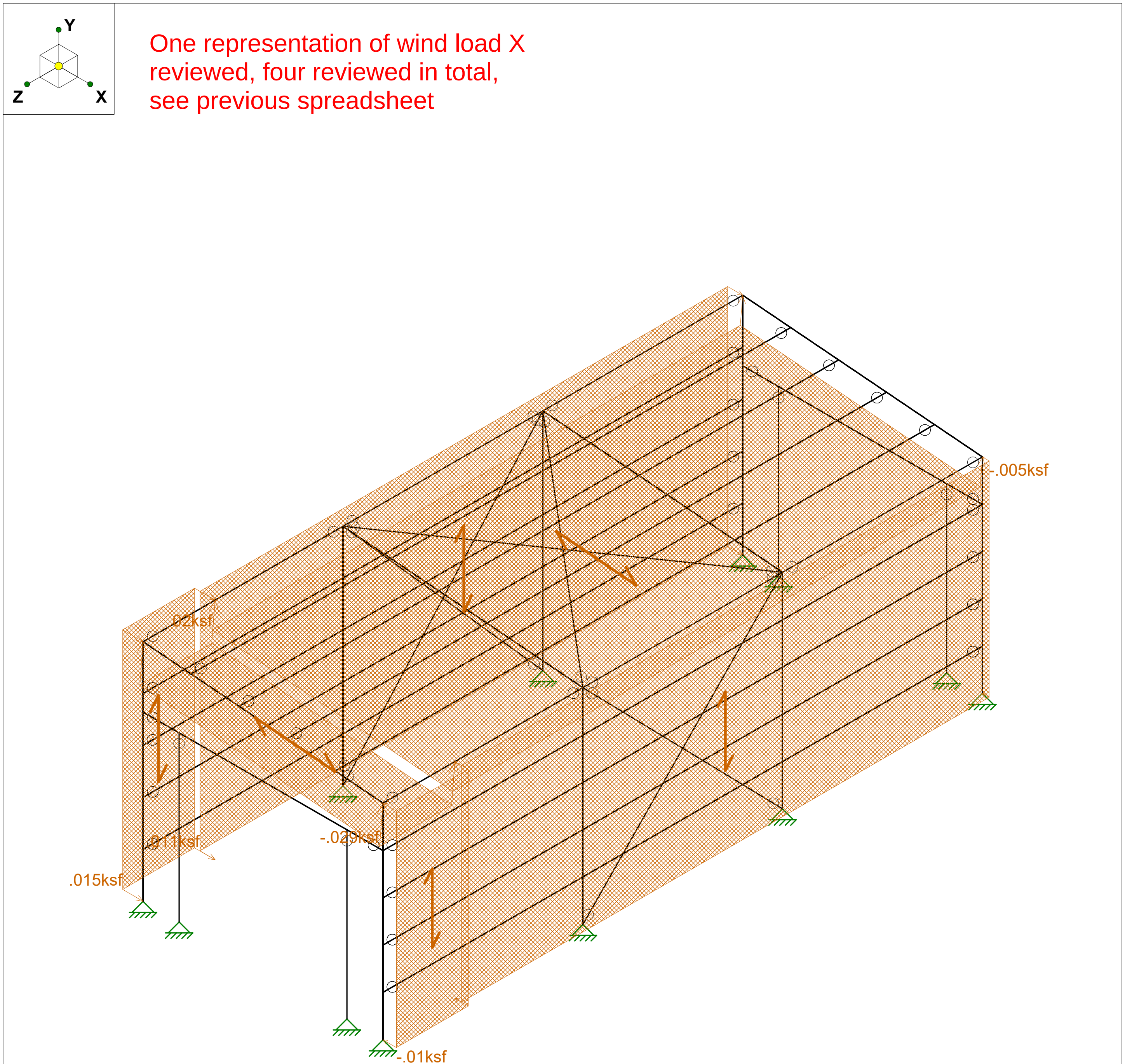
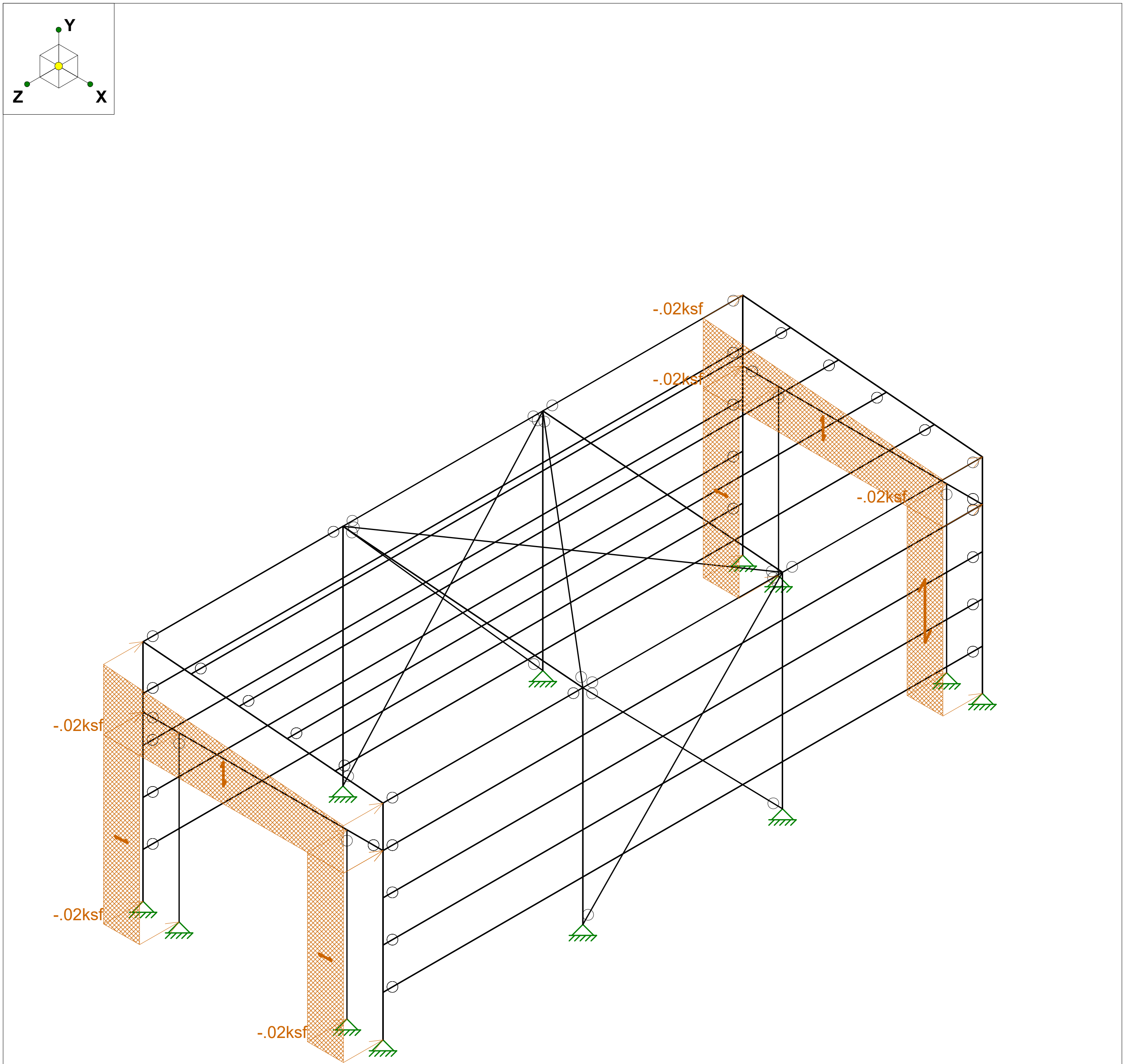
One representation of wind load X reviewed, four reviewed in total, see previous spreadsheet



Loads: BLC 8, x1



Loads: BLC 4, Wind Load Z

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Load Combinations

	Description	Sol..	PD..	SR..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact...
1	ASCE ASD 1	Yes	C		DL	1																		
2	ASCE ASD ...	Yes	C		DL	1	SL	1																
3	ASCE ASD ...	Yes	C		DL	1	W...	.6																
4	ASCE ASD ...	Yes	C		DL	1	WLZ	.6																
5	ASCE ASD ...	Yes	C		DL	1	W...	.45	SL	.75														
6	ASCE ASD ...	Yes	C		DL	1	WLZ	.45	SL	.75														
7	ASCE ASD ...	Yes	C		DL	.6	W...	.6																
8	ASCE ASD ...	Yes	C		DL	.6	WLZ	.6																
9	ASCE ASD ...	Yes	C		DL	1	ELX	.7																
10	ASCE ASD ...	Yes	C		DL	1	ELZ	.7																
11	ASCE ASD ...	Yes	C		DL	1	ELX	-.7																
12	ASCE ASD ...	Yes	C		DL	1	ELZ	-.7																
13	ASCE ASD ...	Yes	C		DL	1	ELX	.525	SL	.75														
14	ASCE ASD ...	Yes	C		DL	1	ELZ	.525	SL	.75														
15	ASCE ASD ...	Yes	C		DL	1	ELX	-.525	SL	.75														
16	ASCE ASD ...	Yes	C		DL	1	ELZ	-.525	SL	.75														
17	ASCE ASD ...	Yes	C		DL	.6	ELX	.7																
18	ASCE ASD ...	Yes	C		DL	.6	ELZ	.7																
19	ASCE ASD ...	Yes	C		DL	.6	ELX	-.7																
20	ASCE ASD ...	Yes	C		DL	.6	ELZ	-.7																
21	ASCE ASD ...	Yes	C		DL	1	OL1	.6																
22	ASCE ASD ...	Yes	C		DL	1	OL1	.45	SL	.75														
23	ASCE ASD ...	Yes	C		DL	.6	OL1	.6																
24	ASCE ASD ...	Yes	C		DL	1	OL2	.6																
25	ASCE ASD ...	Yes	C		DL	1	OL2	.45	SL	.75														
26	ASCE ASD ...	Yes	C		DL	.6	OL2	.6																
27	ASCE ASD ...	Yes	C		DL	1	OL3	.6																
28	ASCE ASD ...	Yes	C		DL	1	OL3	.45	SL	.75														
29	ASCE ASD ...	Yes	C		DL	.6	OL3	.6																

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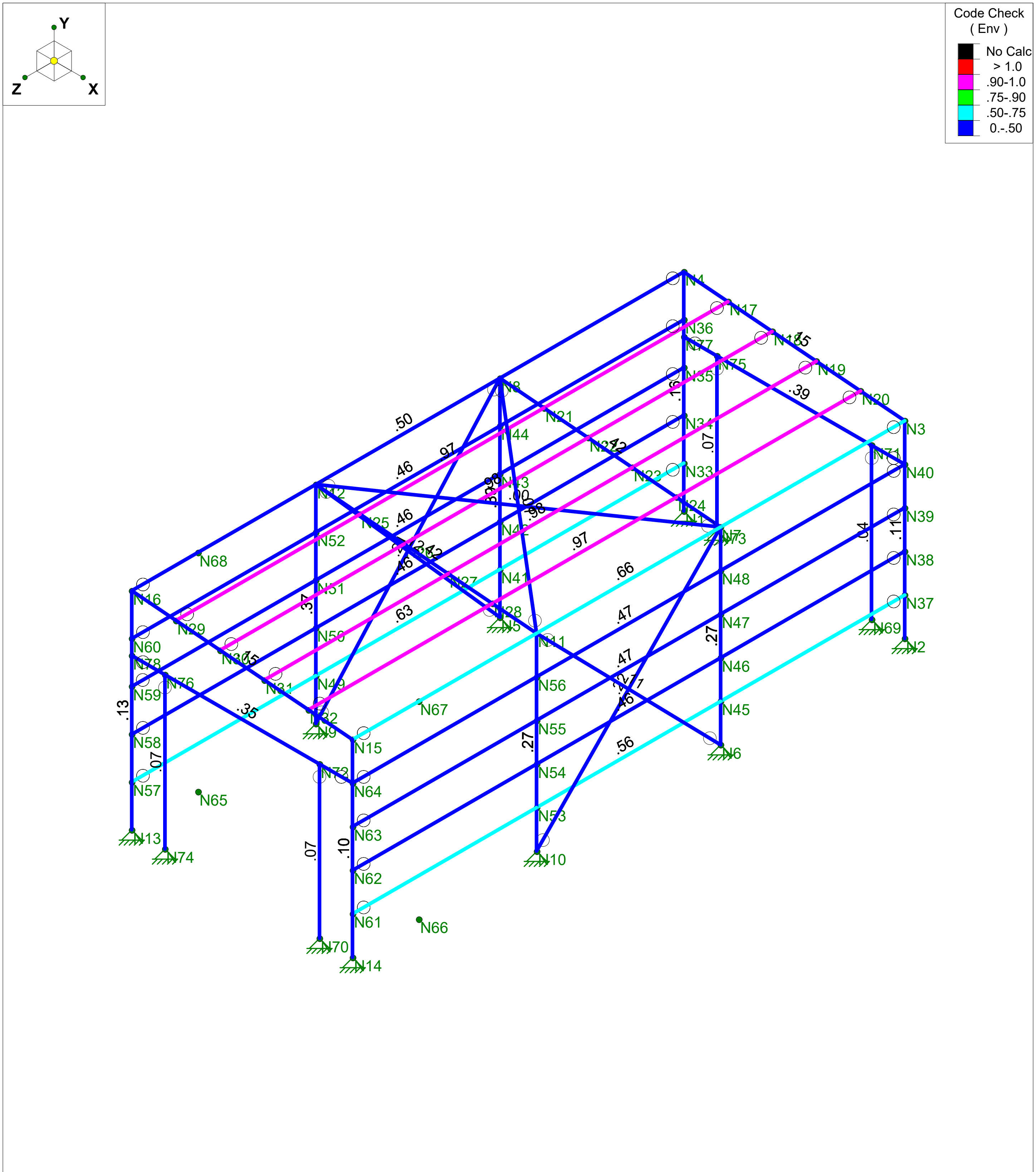

Member Code Checks Displayed (Enveloped)
Envelope Only Solution

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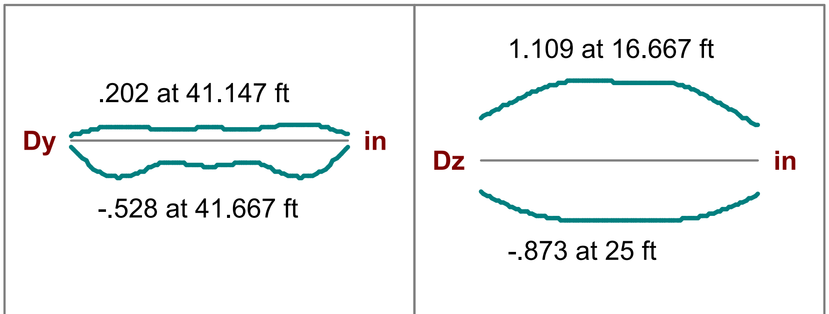
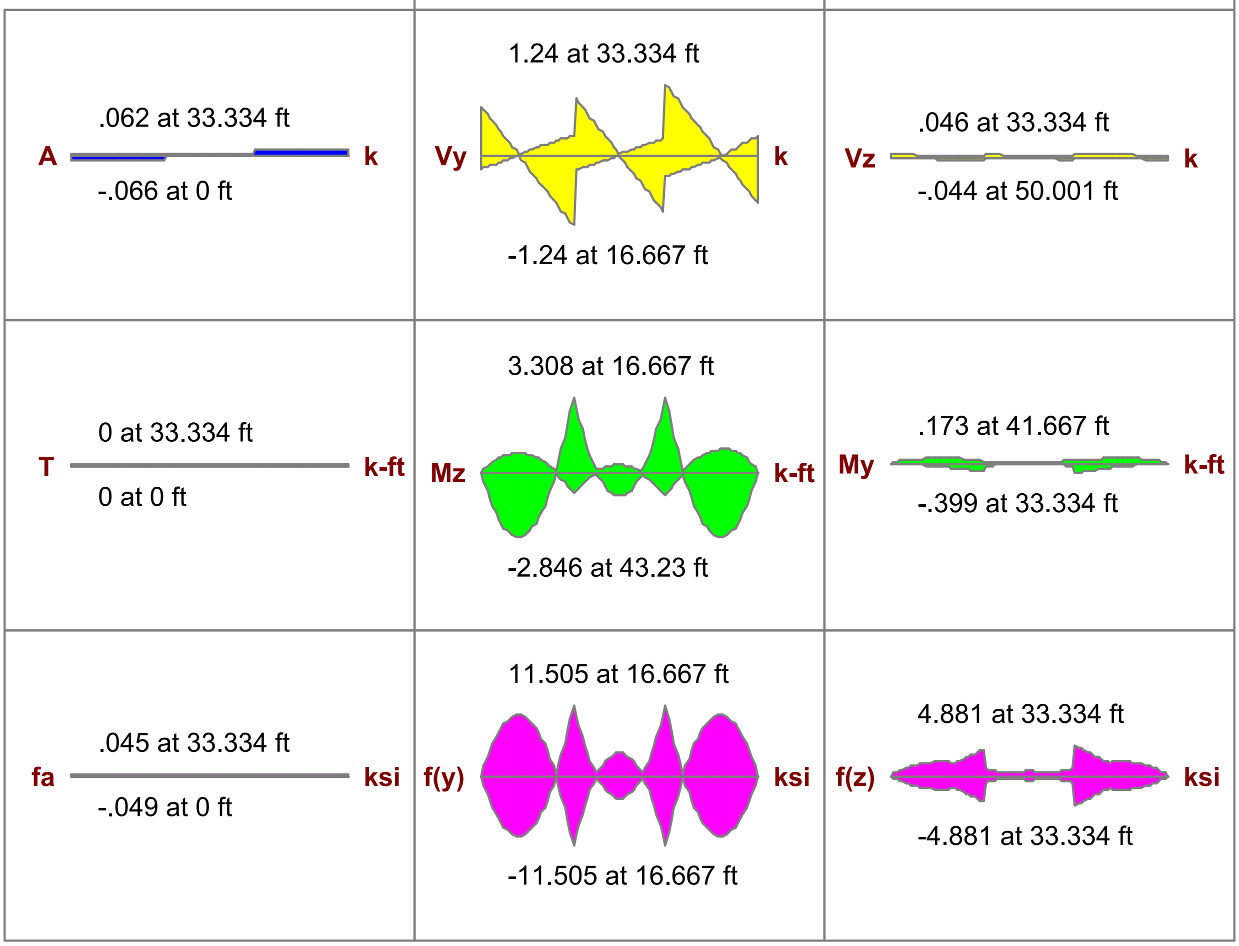
Beam: ***M43***

Shape: **8ZS3.25x085**
Material: **A653 SS Gr50/1**
Length: **50.001 ft**
I Joint: **N19**
J Joint: **N31**
Envelope
Code Check: **0.982 (LC 2)**
Report Based On 97 Sections





AISI S100-16: ASD Code Check

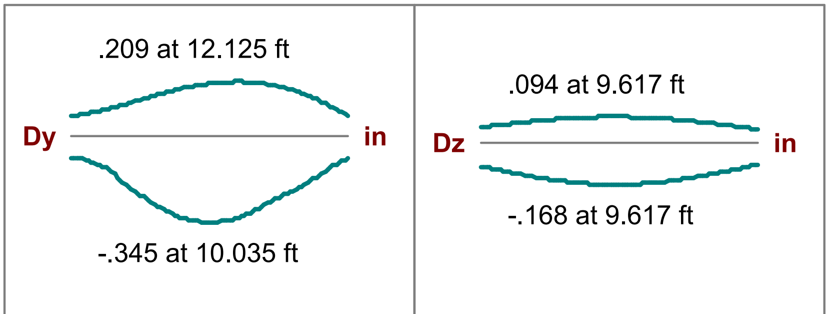
Max Bending Check	**0.982 (LC 2)**	Max Shear Check	**0.170 (y) (LC 2)**	Max Defl Ratio	**L/613**
Location	**16.667 ft**	Location	**16.667 ft**	Location	**42.709 ft**
Equation	**H1.2-1**			Span	**3**
Gov.∅ Equation	**F2/F3**				

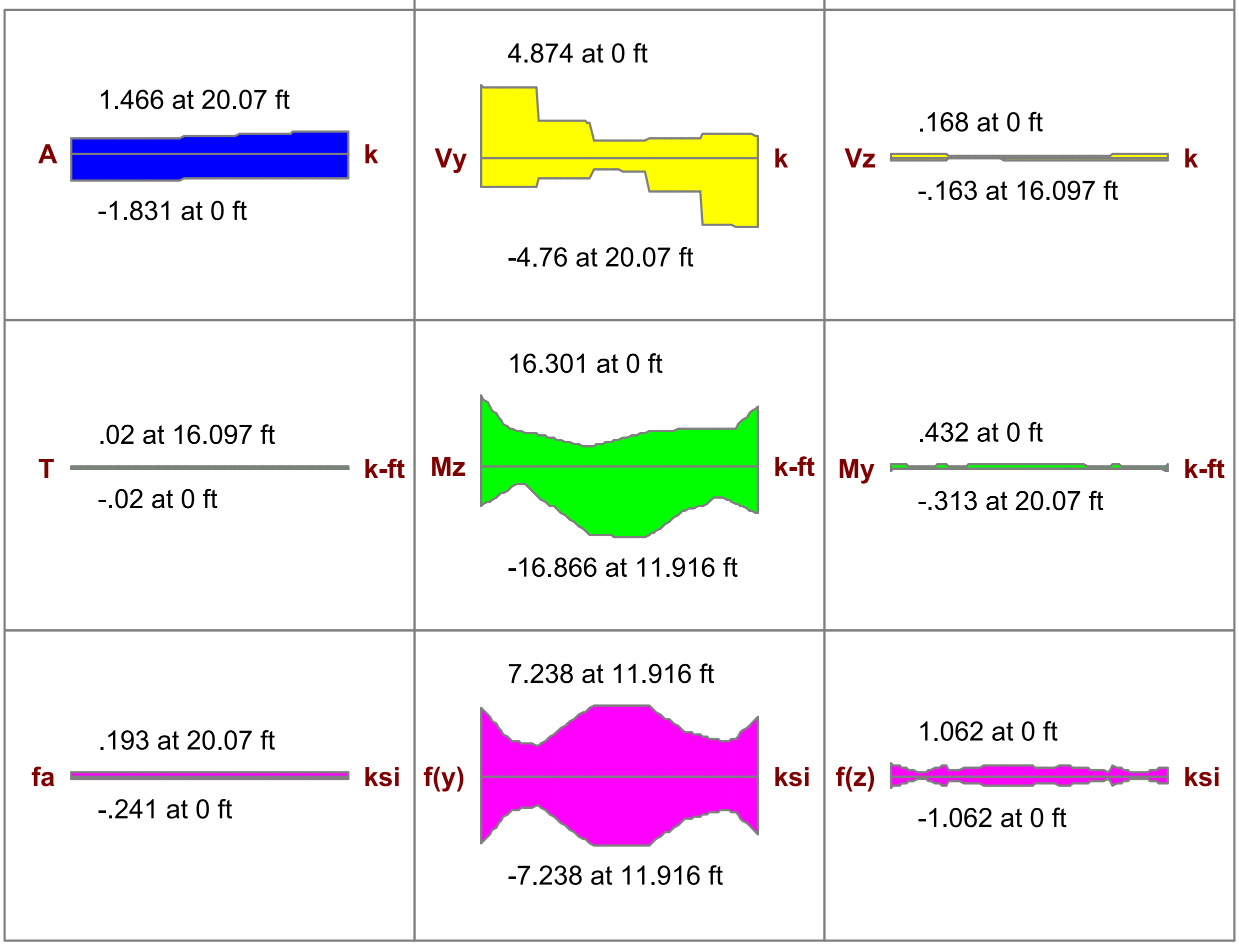
R (I6.2.1) **Not Used**

			y-y	z-z			
Fy	**50 ksi**	Lb	**16.667 ft**	**16.667 ft**		A eff. (Fy)	**.856 in^2**
Pn/Ω	**2.615 k**	KL/r	**196.082**	**196.082**		A eff. (Fn)	**1.36 in^2**
Tn/Ω	**40.719 k**					Iy eff.	**3.089 in^4**
Mny/Ω	**1.939 k-ft**	L Comp Flange		**16.667 ft**		Sy eff. (L)	**.777 in^3**
Mnz/Ω	**3.404 k-ft**	L-torque		**50.001 ft**		Sy eff. (R)	**.84 in^3**
Vny/Ω	**7.33 k**					Iz eff.	**13.768 in^4**
Vnz/Ω	**9.086 k**					Sz eff. (T)	**3.442 in^3**
Cb	**1.444**					Sz eff. (B)	**3.442 in^3**

Beam: *M9*

Shape: **W10X26**
Material: **A992**
Length: **20.07 ft**
I Joint: **N12**
J Joint: **N11**
Envelope
Code Check: **0.418 (LC 2)**
Report Based On 97 Sections





AISC 15th(360-16): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.418 (LC 2)**	Max Shear Check	**0.097 (y) (LC 2)**	Max Defl Ratio	**L/729**
Location	**11.916 ft**	Location	**3.972 ft**	Location	**10.035 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	
Bending Web	**Compact**	Compression Web	**Slender**	**Ae=7.61 in2**

			y-y	z-z
Fy	**50 ksi**	Lb	**20.07 ft**	**20.07 ft**
Pnc/om	**36.539 k**	Lc/r	**176.931**	**55.364**
Pnt/om	**227.844 k**			
Mny/om	**18.713 k-ft**	L Comp Flange	**20.07 ft**	
Mnz/om	**41.373 k-ft**	L-torque	**20.07 ft**	

Smith Monroe Gray
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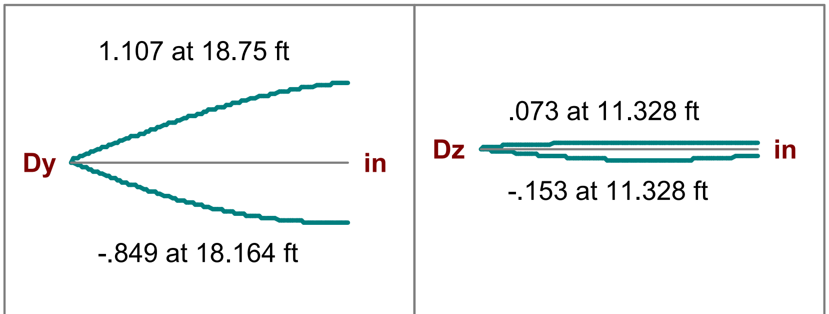
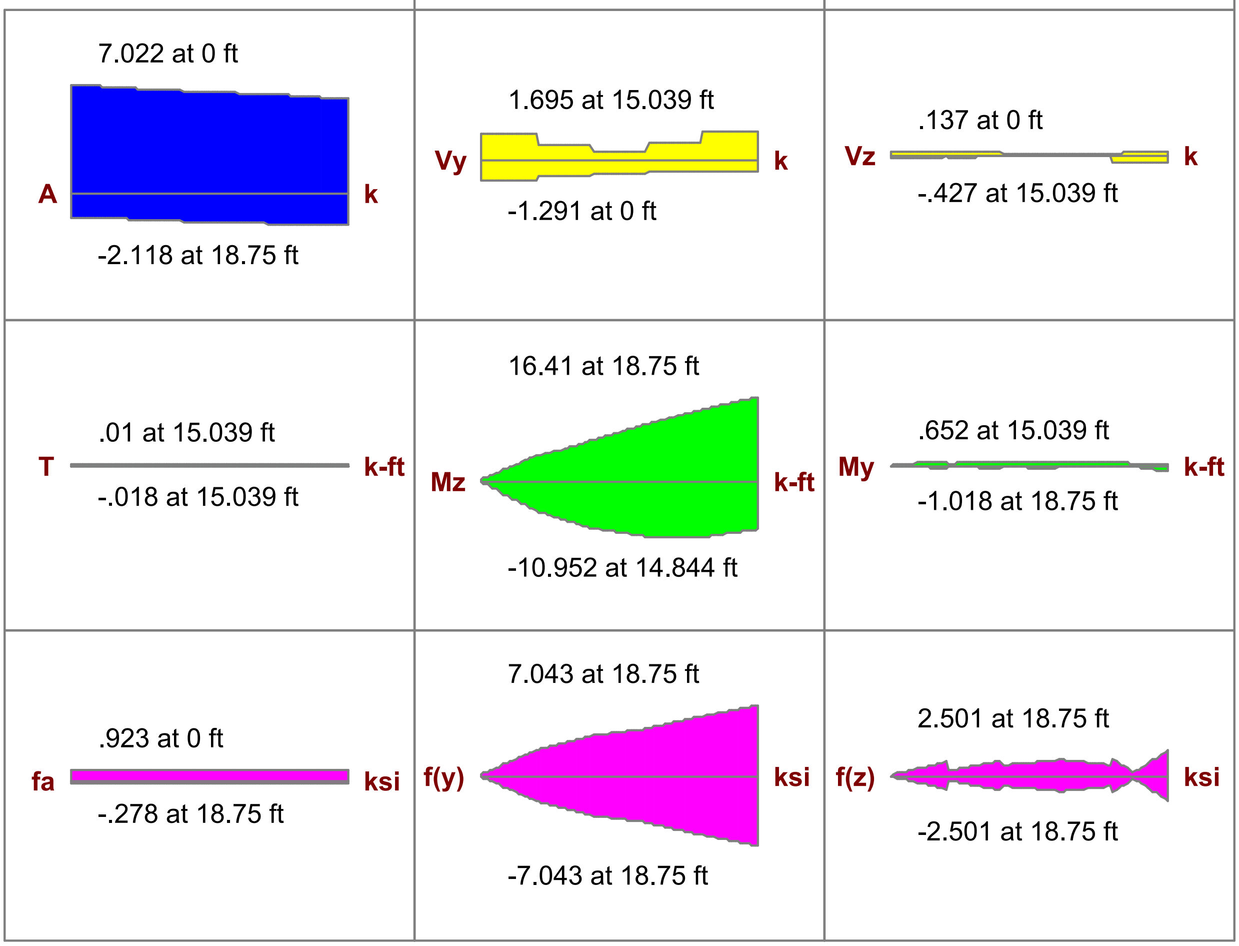
Column: ***M4***

Shape: **W10X26**
Material: **A992**
Length: **18.75 ft**
I Joint: **N5**
J Joint: **N8**
Envelope
Code Check: **0.385 (LC 22)**
Report Based On 97 Sections





AISC 15th(360-16): ASD Code Check

Direct Analysis Method

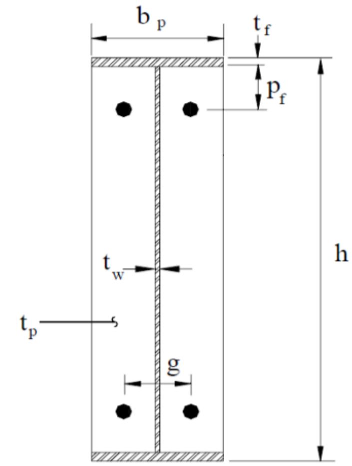
Max Bending Check	**0.385 (LC 22)**	Max Shear Check	**0.035 (y) (LC 26)**
Location	**18.75 ft**	Location	**15.039 ft**
Equation	**H1-1b**	Max Defl Ratio	**L/1048**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	
Bending Web	**Compact**	Compression Web	**Slender**	**Ae=7.61 in2**

			y-y	z-z
Fy	**50 ksi**	Lb	**18.75 ft**	**18.75 ft**
Pnc/om	**41.863 k**	Lc/r	**165.297**	**51.724**
Pnt/om	**227.844 k**			
Mny/om	**18.713 k-ft**	L Comp Flange	**18.75 ft**	
Mnz/om	**54.998 k-ft**	L-torque	**18.75 ft**	

AISC Design Guide 16 - Flush and Extended Multiple Row Moment End Plate Connections

Chapter 3 - Design Procedures



Inputs (ASD Loads)

Geometry Design Data

$b_p = b_f =$	5.75	in
$t_f =$	0.4375	in
$g =$	2.75	in
$p_f =$	1.5	in
$h =$	10.375	in
$M_u = (1.5 \times M_a) =$	28.50	kip-ft
$\gamma_r =$	1.25	
F_t = Tensile Bolt Strength	90.00	ksi
d1 =	8.22	in
$t_p =$	0.63	in
$d_{b\ assumed} =$	0.75	in
$T_b =$	14.00	k
F_{yp} = Plate Yield Strength	36	ksi

Thin Plate Procedure

$s = 1/2\ (b_p\ g\ )^{0.5} =$	1.99	in
$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_f}+\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1(p_f+s)\right]$	50.91	in
$t_{p,req'd} =$	0.51	in
$w' =$	2.06	in
$a_i =$	2.05	in
$F_i' =$	12.08	k
$Q_{max,i} =$	3.16	k
$P_t =$	39.76	k
$\phi M_q =$	451	k-in
	37.6	k-ft

ϕMq > Mu Therefore Plate Thickness OK

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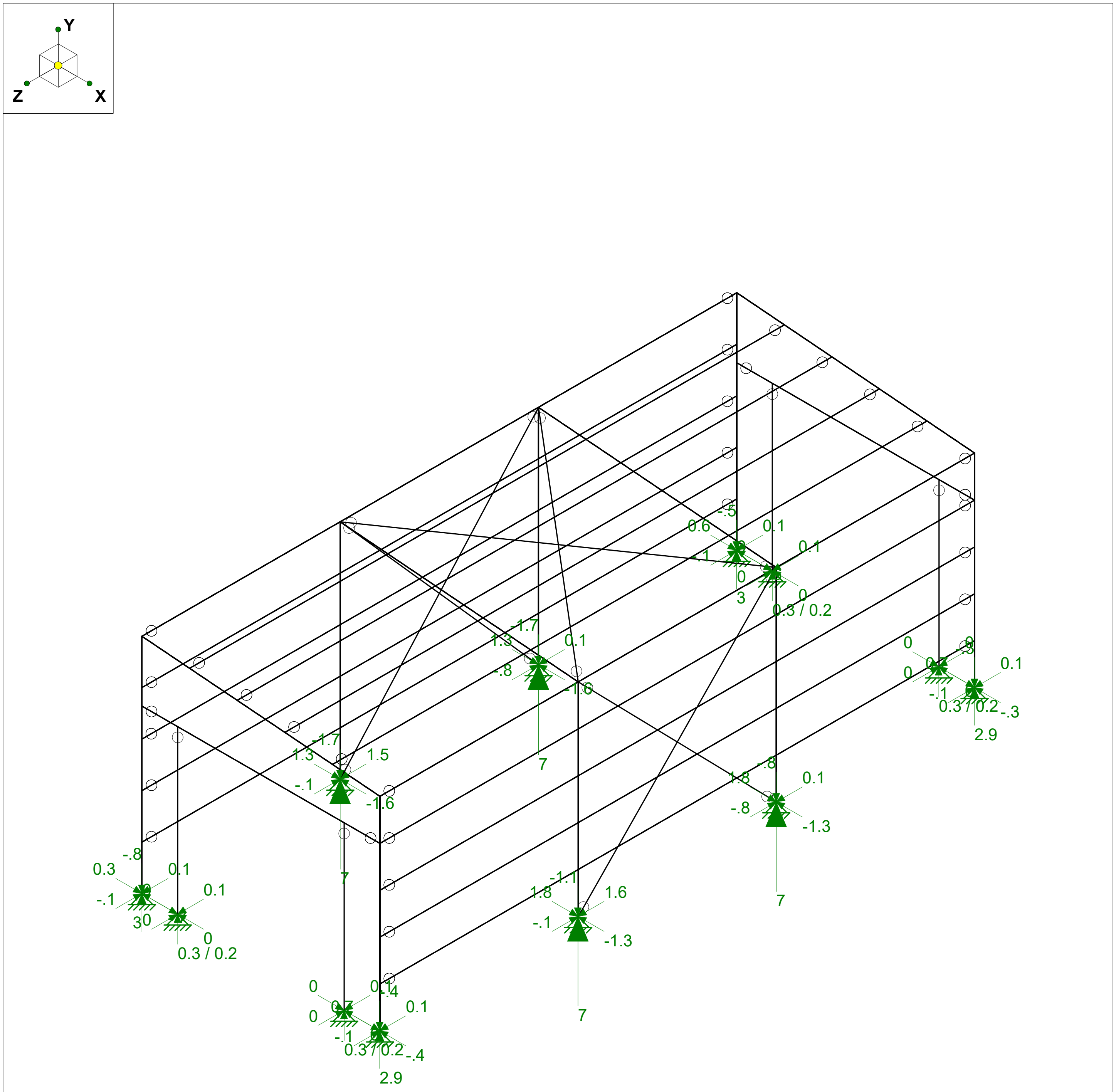


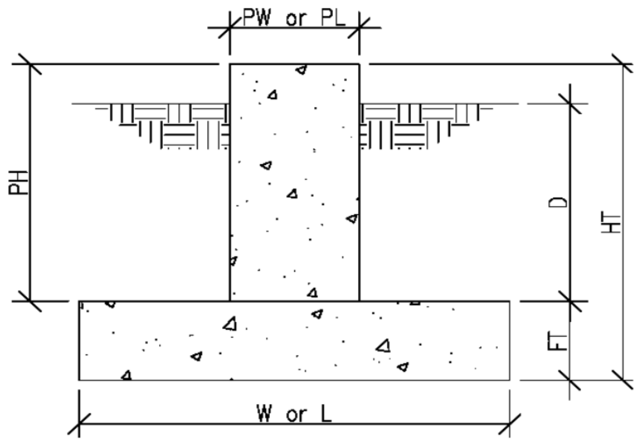
Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	1,500	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.00	
VERTICAL DEAD LOAD =	16.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	0.70	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	4.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	8.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	4.8	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	4.8	KIPS

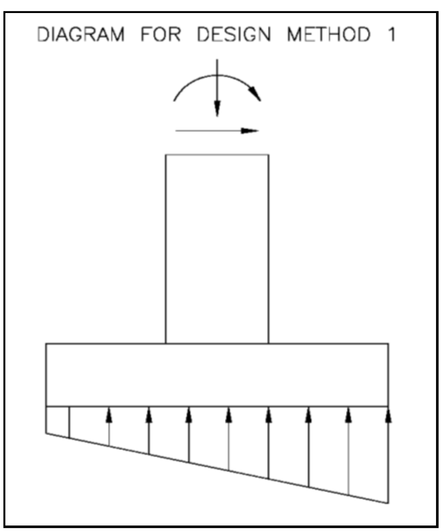
DESIGN METHOD 1

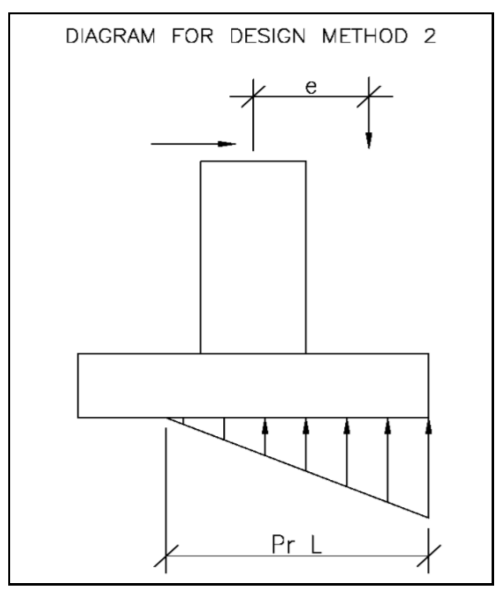
OVERTURNING MOM. =	0.7	FT-KIPS	
SOIL PR. FROM DL =	650.0	PSF	
SOIL PR. FROM MOM. =	(32.8)	PSF	
MIN. PRESSURE =	617.2	PSF	
MAX. PRESSURE =	682.8	PSF	**GOVERNS**

DESIGN METHOD 2

e =	0.03	FT
Pr L =	5.90	FT
MAX. PR =	881.5	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	**650**	PSF	**1,500**	PSF	**OK**
DL + HORIZ. BEARING =	**683**	PSF	**1,500**	PSF	**OK**
F.S. OF OVERTURNING =	**59.43**		**1.5**		**OK**

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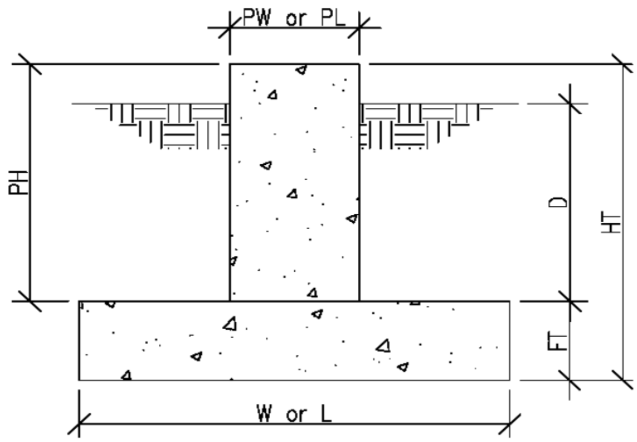
DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	1,500	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.00	
VERTICAL DEAD LOAD =	(1.70)	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	1.60	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	4.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	8.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	4.8	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	4.8	KIPS

DESIGN METHOD 1

OVERTURNING MOM. =	1.6	FT-KIPS	
SOIL PR. FROM DL =	96.9	PSF	
SOIL PR. FROM MOM. =	(75.0)	PSF	
MIN. PRESSURE =	21.9	PSF	
MAX. PRESSURE =	171.9	PSF	**GOVERNS**

DESIGN METHOD 2

e =	0.52	FT
Pr L =	4.45	FT
MAX. PR =	174.1	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING

DIAGRAM FOR DESIGN METHOD 1

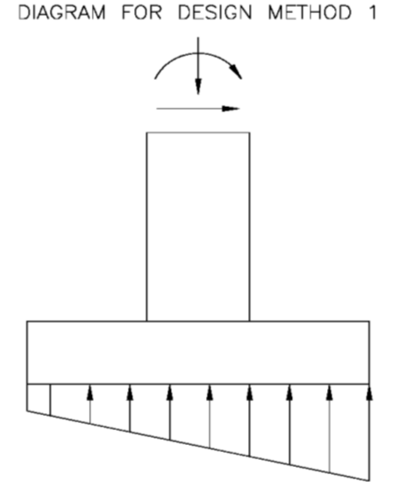
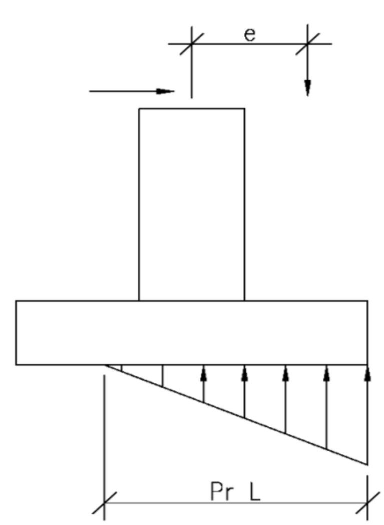


DIAGRAM FOR DESIGN METHOD 2



	ACTUAL		ALLOWABLE		
LL + DL BEARING =	**97**	PSF	**1,500**	PSF	**OK**
DL + HORIZ. BEARING =	**172**	PSF	**1,500**	PSF	**OK**
F.S. OF OVERTURNING =	**3.88**		**1.5**		**OK**

Check foundation reinforcing

$$q_u := 683\ psf \cdot 1.5 = \left(1.025 \cdot 10^{3}\right)\ psf$$

$$M_u := \frac{(2\ ft)^2}{2} \cdot q_u \cdot 1\ ft = 24.588\ kip \cdot in$$

$$A_{s\_min} := 0.0018 \cdot 18\ in \cdot 12\ in = 0.389\ in^2$$

Use two layers of #5 bars at 12

$$A_s := 0.31\ in^2$$

per layer

$$a := \frac{A_s \cdot 60\ ksi}{4\ ksi \cdot 12\ in \cdot 0.85} = 0.456\ in$$

$$\phi M_n := 0.9 \cdot \left(A_s \cdot 60 \cdot ksi \cdot \left(18\ in - 3\ in - \frac{a}{2}\right)\right) = 247.284\ kip \cdot in$$

Reinforcing OK

Check sliding

$$F_s := 1.7\ kip$$

See page 19

$$F_r := 0.5 \cdot (4.8\ kip - 1.7\ kip) + 350\ pcf \cdot \frac{(1.5\ ft)^2}{2} \cdot 8\ ft = 4.7\ kip$$

Sliding Ok

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Phone I Fax:	|	Date:	7/31/2019
E-Mail:			

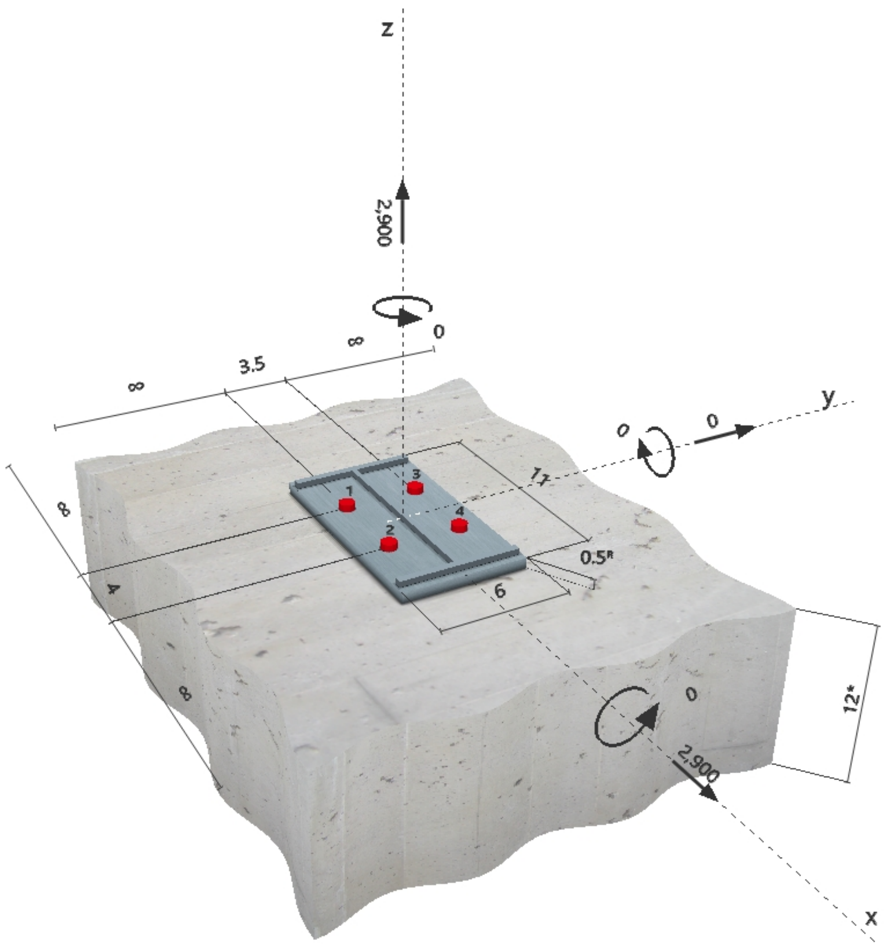
Specifier's comments: Enclose Scale and Loadout

1 Input data

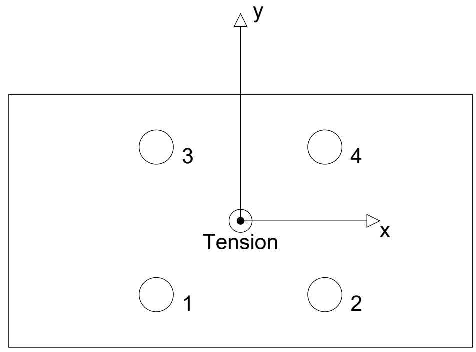
Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-08 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 11.000 in. x 6.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 10.200 in. x 5.750 in. x 0.240 in. x 0.360 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

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

Geometry [in.] & Loading [lb, in.lb]



2 Load case/Resulting anchor forces



Load case: Design loads

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

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	725	725	725	0
2	725	725	725	0
3	725	725	725	0
4	725	725	725	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 2,900 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity $\phi\, N_n$ [lb]	Utilization $\beta_N = N_{ua}/\phi\, N_n$	Status
Steel Strength*	725	14,529	5	OK
Pullout Strength*	725	15,305	5	OK
Concrete Breakout Strength**	2,900	19,743	15	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-08 Eq. (D-3)
$\phi\, N_{sa} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [lb]
19,372

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
19,372	0.750	14,529	725

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-08 Eq. (D-14)
$N_p = 8 A_{brg} f'_c$ ACI 318-08 Eq. (D-15)
$\phi N_{pN} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	f'_c [psi]
1.000	0.91	3,000

Calculations

N_p [lb]
21,864

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
21,864	0.700	15,305	725

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ ACI 318-08 Eq. (D-5)
$\phi N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)
A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)
$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-08 Eq. (D-6)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)
$N_b = k_c\, \lambda\, \sqrt{f'_c}\, h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
0.000	24	1	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]
473.00	324.00	1.000	1.000	1.000	1.000	19,320

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
28,204	0.700	19,743	2,900

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	725	7,555	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,900	39,486	8	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-08 Eq. (D-20)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [lb]
11,623

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
11,623	0.650	7,555	725

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 318-08 Eq. (D-31)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\, \lambda\, \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

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

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f_c' [psi]
1.000	-	24	1	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]
473.00	324.00	1.000	1.000	1.000	1.000	19,320

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
56,409	0.700	39,486	2,900

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5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.147	0.096	5/3	7	OK

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

6 Warnings

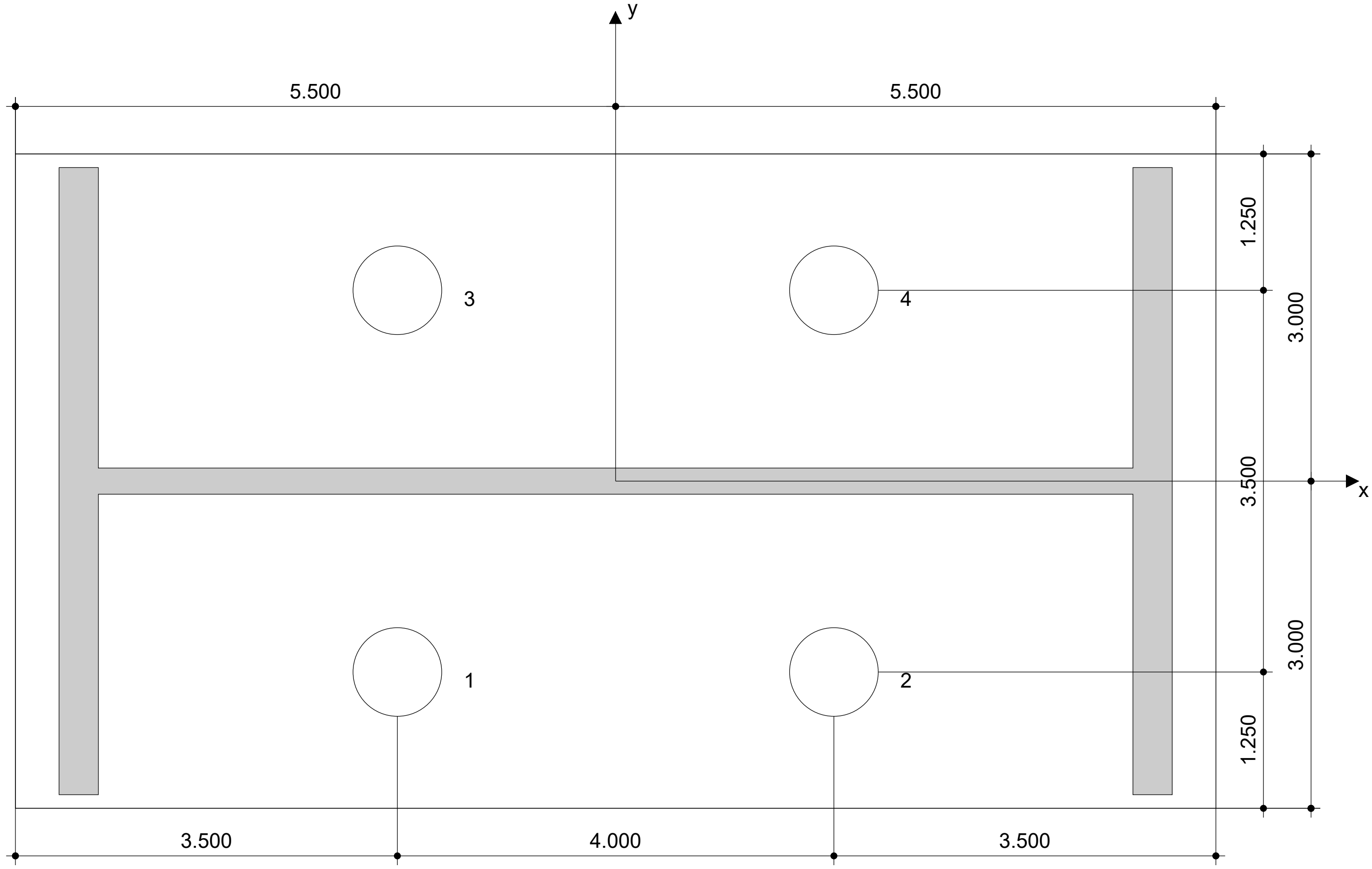
- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor 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 Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); (L x W x T x FT) = 10.200 in. x 5.750 in. x 0.240 in. x 0.360 in.
Hole diameter in the fixture: d_f = 0.813 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.000 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.000	-1.750	-	-	-	-
2	2.000	-1.750	-	-	-	-
3	-2.000	1.750	-	-	-	-
4	2.000	1.750	-	-	-	-

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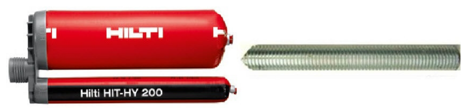
Phone I Fax:	|	Date:	7/31/2019
E-Mail:			

Specifier's comments: Enclose Scale and Loadout

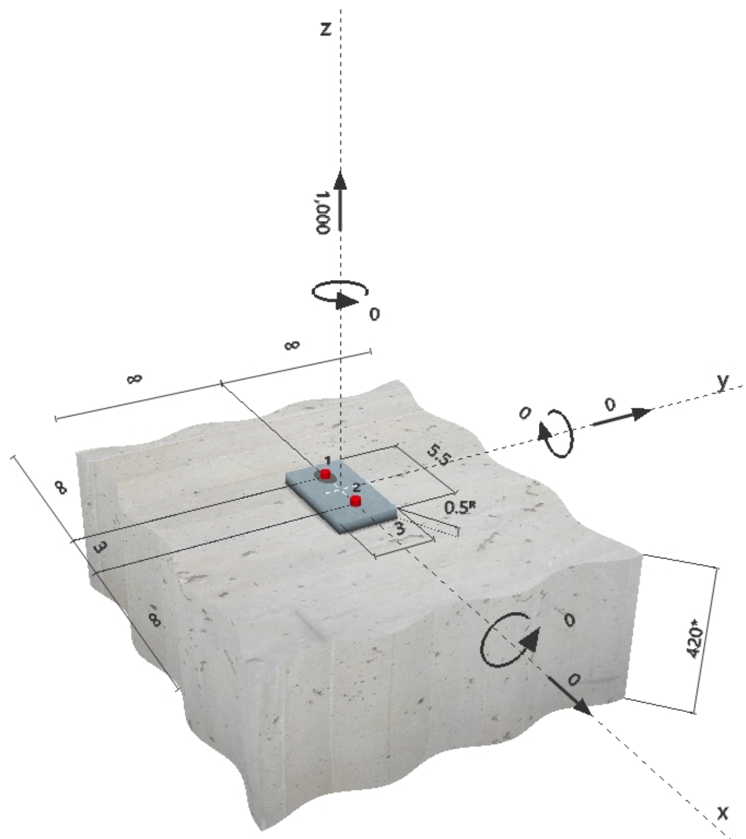
1 Input data



Anchor type and diameter:	**HIT-HY 200 + HAS-V-36 (ASTM F1554 Gr.36) 1/2**
Effective embedment depth:	$h_{ef,act}$ = 6.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 1554 Grade 36
Evaluation Service Report:	ESR-3187
Issued I Valid:	3/1/2018 | 3/1/2020
Proof:	Design method ACI 318-08 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 5.500 in. x 3.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 420.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

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

Geometry [in.] & Loading [lb, in.lb]



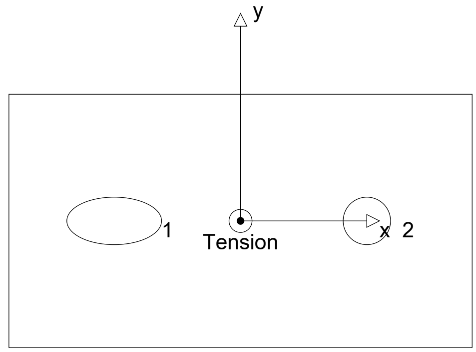
Company: SMG
Specifier: JWC
Address:
Phone I Fax: |
E-Mail:

2 Load case/Resulting anchor forces

Load case: Design loads

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

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	500	0	0	0
2	500	0	0	0



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 1,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 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*	500	6,173	9	OK
Bond Strength**	1,000	8,428	12	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	1,000	9,473	11	OK

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

N_{sa} [lb]
8,230

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
8,230	0.750	6,173	500

3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na}\ \psi_{ec2,Na}\ \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-11 Eq. (D-19)

$\phi\ N_{ag} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Na} = see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-11 Eq. (D-20)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-11 Eq. (D-21)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N'}{c_{Na}}}\right) \leq 1.0$ ACI 318-11 Eq. (D-23)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-11 Eq. (D-25)

$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-27)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-11 Eq. (D-22)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2,220	0.500	6.000	∞	1,135

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
0.000	0.000	10.171	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.071	242.43	200.00	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	1.000	1.000	10,697

Results

N_{ag} [lb]	ϕ_{bond}	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
12,966	0.650	8,428	1,000

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ ACI 318-08 Eq. (D-5)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\, \lambda\, \sqrt{f'_c}\, h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
10.171	17	1	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
378.00	324.00	1.000	1.000	1.000	1.000	12,492

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
14,574	0.650	9,473	1,000

Company:	SMG	Page:	5
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	7/31/2019
E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	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 (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (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 Anchor 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 Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions
- The ACI 318-08 version of the software does not account for adhesive anchor special design provisions corresponding to overhead applications.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

Company:	SMG	Page:	6
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	7/31/2019
E-Mail:			

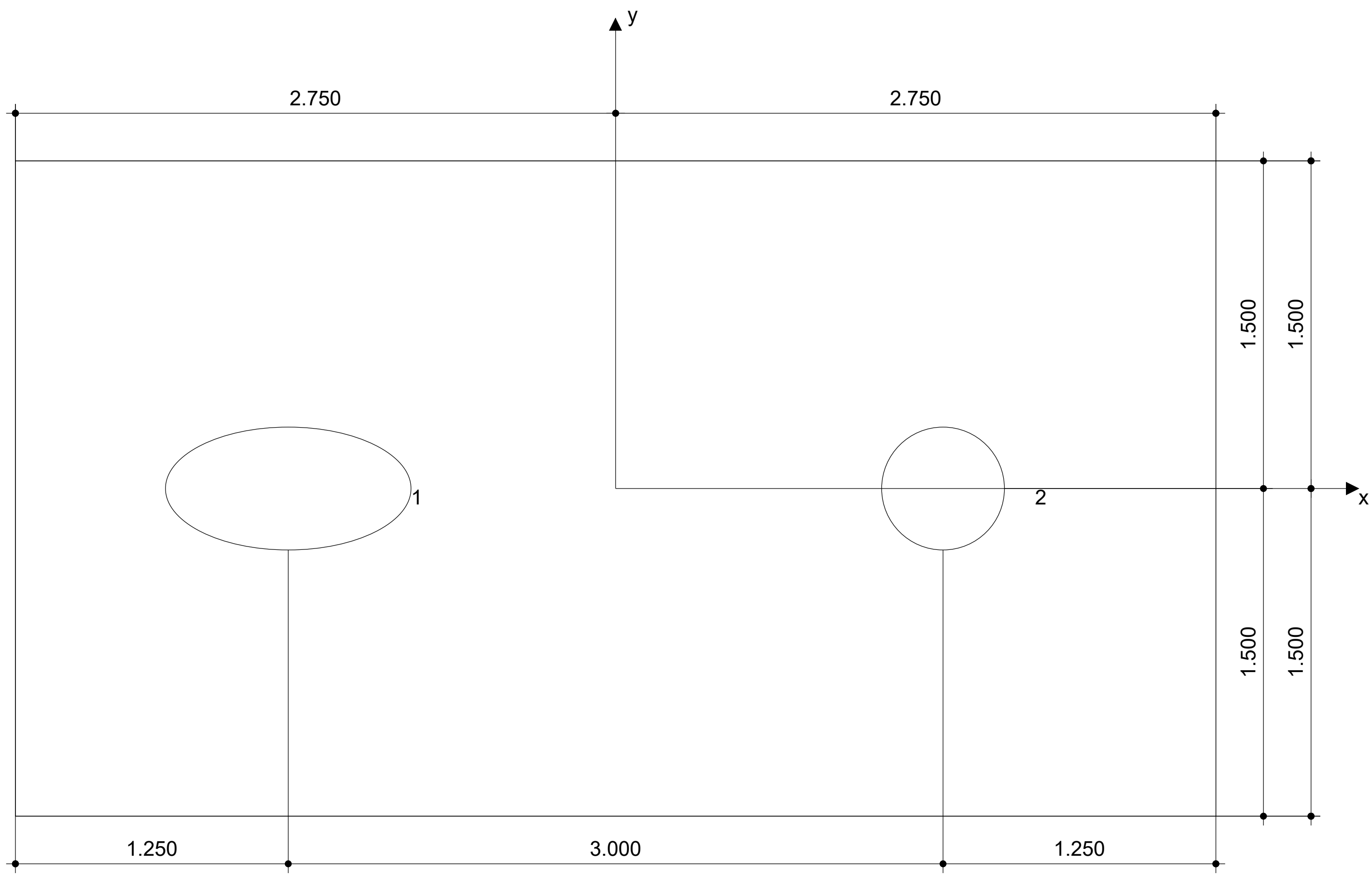
6 Installation data

Anchor plate, steel: -
Profile: no profile
Hole diameter in the fixture: d_f = 0.563 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-V-36 (ASTM F1554 Gr.36) 1/2
Installation torque: 360.001 in.lb
Hole diameter in the base material: 0.563 in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.250 in.

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.500	0.000	-	-	-	-
2	1.500	0.000	-	-	-	-

Company:	SMG	Page:	7
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	7/31/2019
E-Mail:			

7 Remarks; Your Cooperation Duties

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TABLE 9 - ALLOWABLE UNIFORM LOADS (psf) FOR VERCO STEEL DECK PANELS WITHOUT CONCRETE FILL[1,2,3]

SPAN	DECK GAGE	CRITERIA	2'-0"	3'-0"	4'-0"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	11'-0"	12'-0"
			PLB™-36 & HSB®-36 and PLB™ & B FORMLOK™															
SINGLE	22	Stress	300	300	220	141	116	98	83	72	63	55	49	43	39	35	29	24
		L/360	♦♦♦	287	121	62	47	36	28	23	18	15	13	11	9	8	6	4
		L/240	♦♦♦	♦♦♦	182	93	70	54	42	34	28	23	19	16	14	12	9	7
		L/180	♦♦♦	♦♦♦	♦♦♦	124	93	72	56	45	37	30	25	21	18	15	12	9
	20	Stress	300	300	288	184	152	128	109	94	82	72	64	57	51	46	38	32
		L/360	♦♦♦	♦♦♦	150	77	58	44	35	28	23	19	16	13	11	10	7	6
		L/240	♦♦♦	♦♦♦	225	115	86	67	52	42	34	28	23	20	17	14	11	8
		L/180	♦♦♦	♦♦♦	♦♦♦	153	115	89	70	56	45	37	31	26	22	19	14	11
	18	Stress	300	300	300	251	208	174	149	128	112	98	87	78	70	63	52	44
		L/360	♦♦♦	♦♦♦	207	106	79	61	48	39	31	26	22	18	15	13	10	8
		L/240	♦♦♦	♦♦♦	♦♦♦	159	119	92	72	58	47	39	32	27	23	20	15	11
		L/180	♦♦♦	♦♦♦	♦♦♦	212	159	122	96	77	63	52	43	36	31	26	20	15
	16	Stress	300	300	300	300	264	222	189	163	142	125	110	99	88	80	66	55
		L/360	♦♦♦	♦♦♦	261	133	100	77	61	49	40	33	27	23	19	17	13	10
		L/240	♦♦♦	♦♦♦	♦♦♦	200	150	116	91	73	59	49	41	34	29	25	19	14
		L/180	♦♦♦	♦♦♦	♦♦♦	267	200	154	121	97	79	65	54	46	39	33	25	19
DOUBLE	22	Stress	300	300	235	150	124	104	89	77	67	59	52	46	42	38	31	26
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	122	94	74	59	48	40	33	28	24	20	15	12
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	49	42	35	30	23	18
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	30	23
	20	Stress	300	300	296	190	157	132	112	97	84	74	66	59	53	47	39	33
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	146	113	89	71	58	48	40	33	28	24	18	14
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	71	59	50	43	37	27	21
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	37	28
	18	Stress	300	300	300	265	219	184	157	135	118	103	92	82	73	66	55	46
		L/360	♦♦♦	♦♦♦	♦♦♦	258	194	149	117	94	76	63	53	44	38	32	24	19
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	115	94	79	66	56	48	36	28
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	64	48	37
	16	Stress	300	300	300	300	271	228	194	167	146	128	113	101	91	82	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	241	186	146	117	95	78	65	55	47	40	30	23
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	118	98	83	70	60	45	35
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	80	60	46
TRIPLE	22	Stress	300	300	294	188	155	131	111	96	84	73	65	58	52	47	39	33
		L/360	♦♦♦	♦♦♦	247	127	95	73	58	46	38	31	26	22	18	16	12	9
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	110	86	69	56	46	39	33	28	24	18	14
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	92	75	62	52	43	37	32	24	18
	20	Stress	300	300	300	237	196	165	140	121	105	93	82	73	66	59	49	41
		L/360	♦♦♦	♦♦♦	298	152	115	88	69	56	45	37	31	26	22	19	14	11
		L/240	♦♦♦	♦♦♦	♦♦♦	229	172	132	104	83	68	56	47	39	33	29	21	17
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	139	111	90	74	62	52	44	38	29	22
	18	Stress	300	300	300	300	274	230	196	169	147	129	115	102	92	83	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	202	152	117	92	74	60	49	41	35	29	25	19	15
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	228	175	138	110	90	74	62	52	44	38	28	22
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	184	147	120	99	82	69	59	50	38	29
	16	Stress	300	300	300	300	300	285	243	209	182	160	142	127	114	103	85	71
		L/360	♦♦♦	♦♦♦	♦♦♦	251	189	145	114	92	74	61	51	43	37	31	24	18
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	283	218	172	137	112	92	77	65	55	47	35	27
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	229	183	149	123	102	86	73	63	47	36

Page 24 has the footnotes.

(continued)

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MAPLE VALLEY
NEW LOADOUT ENCLOSURE

ENGINEER	STRUCTURAL CALCULATIONS	DATE
JWC	FOR	11/1/19

LAKESIDE INDUSTRIES

PROJECT NO.		REVISION
18-183F	**APPENDED CALCS**	0

CONTENTS: SHT. NO.

DESIGN REFERENCES:

ASCE 7 MINIMUM DESIGN LOADS FOR BUILDING AND OTHER STRUCTURES
AISC STEEL CONSTRUCTION MANUAL
ACI 318 BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE

Project Scope

These additional calculations are developed based on the 1st review comments

Foundation Design

Develop reactions to end bay, note that these are max envelope reaction and are conservative as they do not all happen concurrently

$P := 3000\ lbf$	LC 2, Dead + 0.6WLX
$T := 800\ lbf$	LC 7 0.6DL + 0.6WLX
$V_x := 700\ lbf$	LC 21 DL + 0.6 Other WLX
$V_y := 100\ lbf$	LC 4 DL + 0.6 WLZ

As can be seen in the following 2 spreadsheets, that the corner condition is not controlling, foundation design ok

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

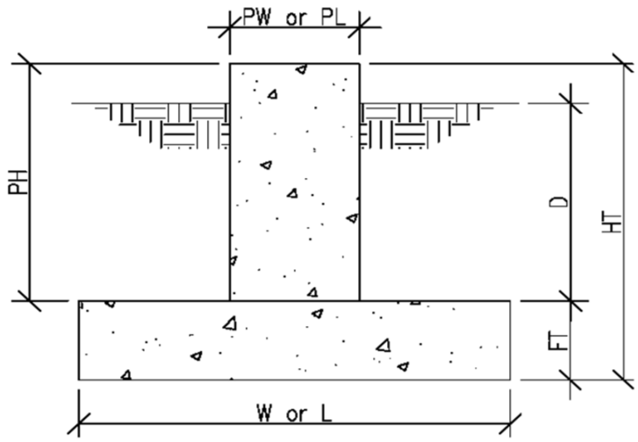
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	1,500	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.00	
VERTICAL DEAD LOAD =	(0.80)	KIPS
VERTICAL LIVE LOAD =	0.70	KIPS
HORIZONTAL LOAD =	0.00	KIPS
MOMENT @ TOP OF FOOTING =	3.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	6.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	3.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	2.7	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	2.7	KIPS



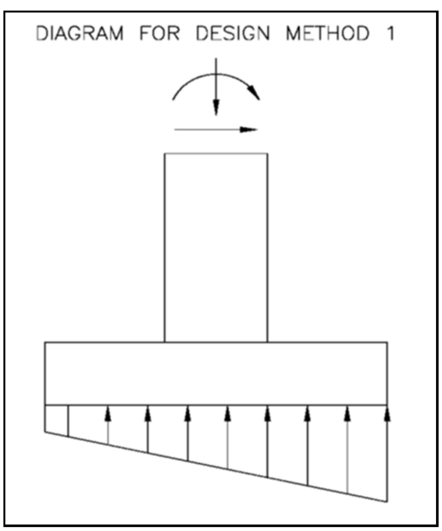
DESIGN METHOD 1

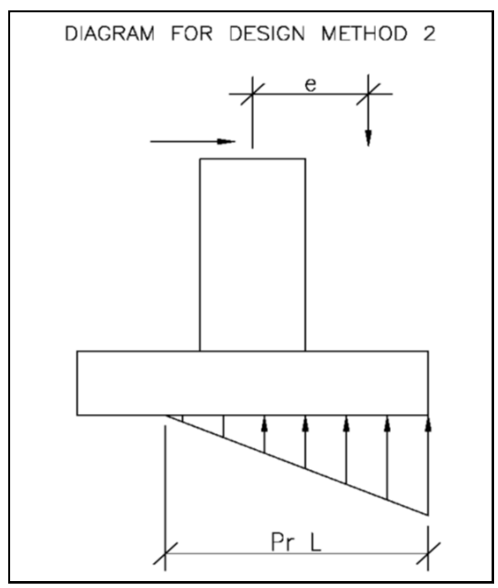
OVERTURNING MOM. =	3.0	FT-KIPS
SOIL PR. FROM DL =	105.6	PSF
SOIL PR. FROM MOM. =	(166.7)	PSF
MIN. PRESSURE =	(61.1)	PSF
MAX. PRESSURE =	272.2	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	1.58	FT	
Pr L =	4.26	FT	
MAX. PR =	297.1	PSF	**<--- GOVERNS**





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	**144**	PSF	**1,500**	PSF	**OK**
DL + HORIZ. BEARING =	**297**	PSF	**1,500**	PSF	**OK**
F.S. OF OVERTURNING =	**1.90**		**1.5**		**OK**

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

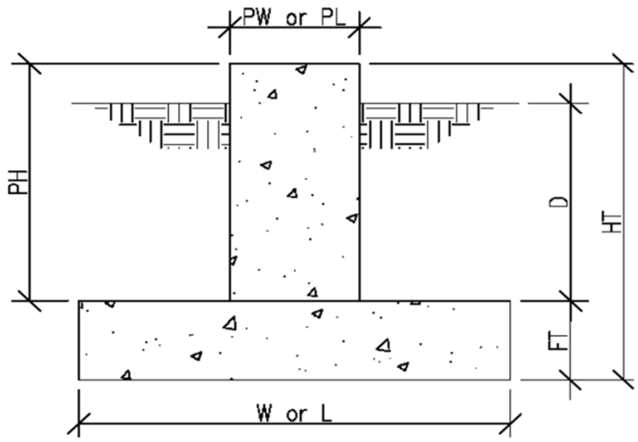
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	1,500	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.00	
VERTICAL DEAD LOAD =	3.00	KIPS
VERTICAL LIVE LOAD =	0.70	KIPS
HORIZONTAL LOAD =	0.00	KIPS
MOMENT @ TOP OF FOOTING =	3.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	6.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	3.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	2.7	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	2.7	KIPS



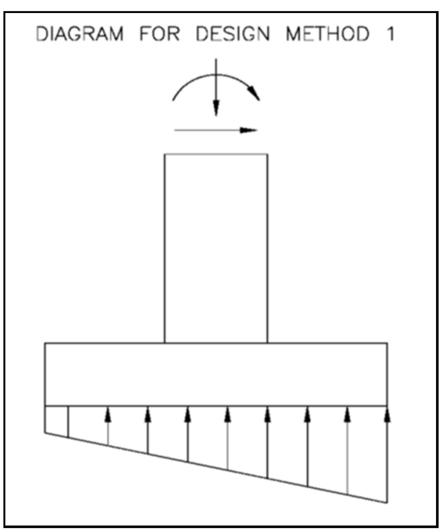
DESIGN METHOD 1

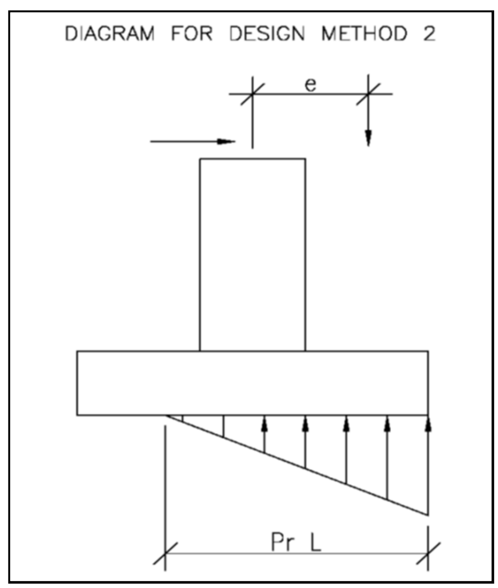
OVERTURNING MOM. =	3.0	FT-KIPS	
SOIL PR. FROM DL =	316.7	PSF	
SOIL PR. FROM MOM. =	(166.7)	PSF	
MIN. PRESSURE =	150.0	PSF	
MAX. PRESSURE =	483.3	PSF	**GOVERNS**

DESIGN METHOD 2

e =	0.53	FT
Pr L =	7.42	FT
MAX. PR =	512.1	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	**356**	PSF	**1,500**	PSF	**OK**
DL + HORIZ. BEARING =	**483**	PSF	**1,500**	PSF	**OK**
F.S. OF OVERTURNING =	**5.70**		**1.5**		**OK**

The following RISA output is for LRFD seismic load combinations only.

$\Omega_{0\_z} := 2$ Overstrength in the Z direction

$\Omega_{0\_x} := 3$ Overstrength in the X direction

Load Combinations

	Description	Sol...	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 1		C		DL	1										
2	ASCE ASD 3 (b)		C		DL	1	SL	1								
3	ASCE ASD 5 (a) (a)		C		DL	1	WLX	.6								
4	ASCE ASD 5 (a) (b)		C		DL	1	WLZ	.6								
5	ASCE ASD 6 (c) (a)		C		DL	1	WLX	.45	SL	.75						
6	ASCE ASD 6 (c) (b)		C		DL	1	WLZ	.45	SL	.75						
7	ASCE ASD 7 (a)		C		DL	.6	WLX	.6								
8	ASCE ASD 7 (b)		C		DL	.6	WLZ	.6								
9	ASCE ASD 5 (b) (a)		C		DL	1	ELX	.7								
10	ASCE ASD 5 (b) (b)		C		DL	1	ELZ	.7								
11	ASCE ASD 5 (b) (c)		C		DL	1	ELX	-.7								
12	ASCE ASD 5 (b) (d)		C		DL	1	ELZ	-.7								
13	ASCE ASD 6 (d) (a)		C		DL	1	ELX	.525	SL	.75						
14	ASCE ASD 6 (d) (b)		C		DL	1	ELZ	.525	SL	.75						
15	ASCE ASD 6 (d) (c)		C		DL	1	ELX	-.525	SL	.75						
16	ASCE ASD 6 (d) (d)		C		DL	1	ELZ	-.525	SL	.75						
17	ASCE ASD 8 (a)		C		DL	.6	ELX	.7								
18	ASCE ASD 8 (b)		C		DL	.6	ELZ	.7								
19	ASCE ASD 8 (c)		C		DL	.6	ELX	-.7								
20	ASCE ASD 8 (d)		C		DL	.6	ELZ	-.7								
21	ASCE ASD 5 (a) (a)		C		DL	1	OL1	.6								
22	ASCE ASD 6 (c) (a)		C		DL	1	OL1	.45	SL	.75						
23	ASCE ASD 7 (a)		C		DL	.6	OL1	.6								
24	ASCE ASD 5 (a) (a)		C		DL	1	OL2	.6								
25	ASCE ASD 6 (c) (a)		C		DL	1	OL2	.45	SL	.75						
26	ASCE ASD 7 (a)		C		DL	.6	OL2	.6								
27	ASCE ASD 5 (a) (a)		C		DL	1	OL3	.6								
28	ASCE ASD 6 (c) (a)		C		DL	1	OL3	.45	SL	.75						
29	ASCE ASD 7 (a)		C		DL	.6	OL3	.6								
30	For Anchor Design O...															
31	ASCE Strength 5 (os-a)	Yes	Y		DL	1.2	Om*ELX	1	LL	.5	LLS	1	SL	.2	SLN	.2
32	ASCE Strength 5 (os-b)	Yes	Y		DL	1.2	Om*ELZ	1	LL	.5	LLS	1	SL	.2	SLN	.2
33	ASCE Strength 5 (os-c)	Yes	Y		DL	1.2	Om*ELX	-1	LL	.5	LLS	1	SL	.2	SLN	.2
34	ASCE Strength 5 (os-d)	Yes	Y		DL	1.2	Om*ELZ	-1	LL	.5	LLS	1	SL	.2	SLN	.2
35	ASCE Strength 7 (os-a)	Yes	Y		DL	.9	Om*ELX	1								
36	ASCE Strength 7 (os-b)	Yes	Y		DL	.9	Om*ELZ	1								
37	ASCE Strength 7 (os-c)	Yes	Y		DL	.9	Om*ELX	-1								
38	ASCE Strength 7 (os-d)	Yes	Y		DL	.9	Om*ELZ	-1								

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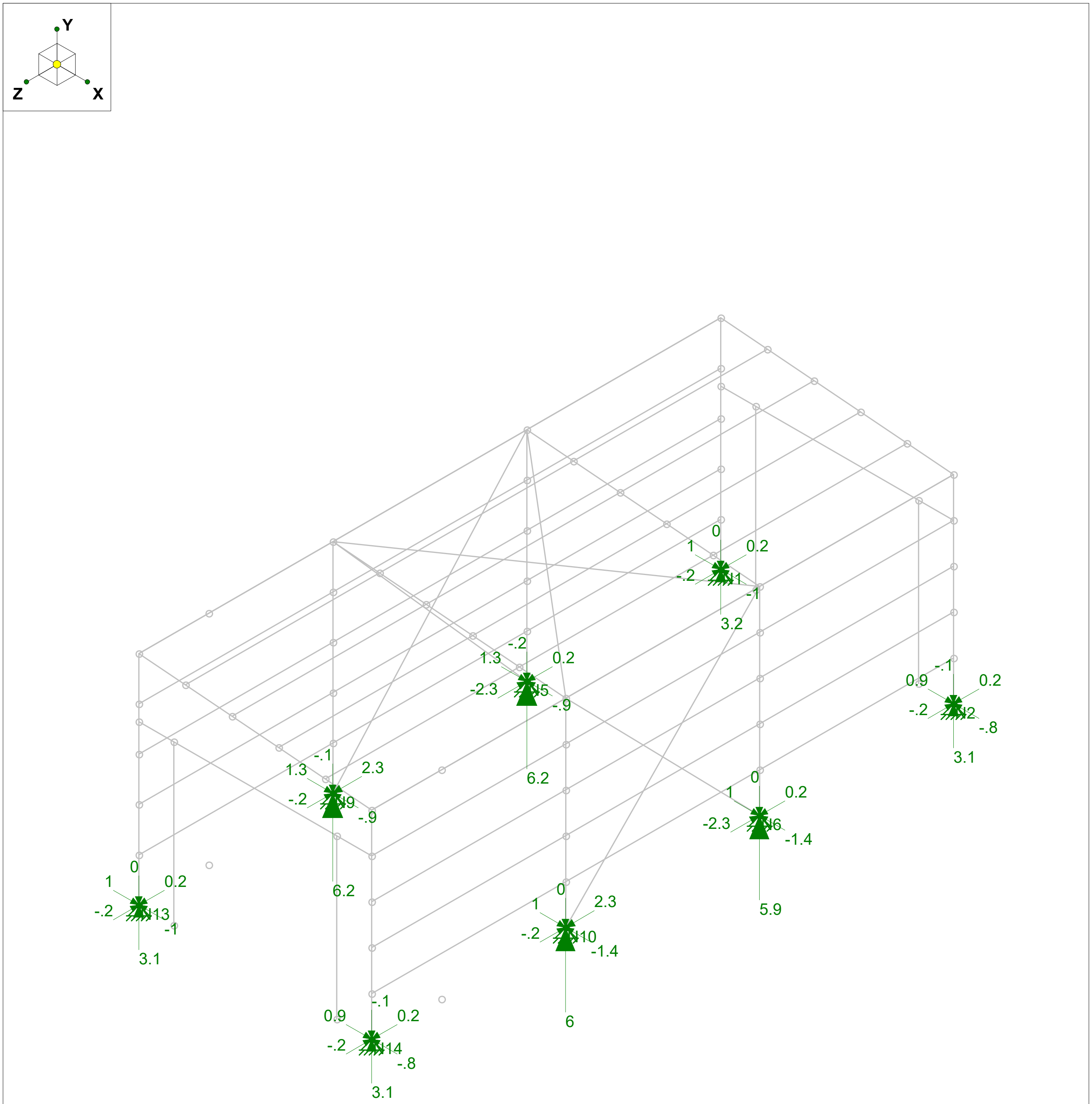
PROJECT Maple Valley Asphalt Plant

New Loadout Enclosure - Appended Calcs

BY JWC DATE 11/01/19 REV. 1

JOB NO. 18-183F SHEET 7 OF 22



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

Envelope Joint Reactions - Overstrength

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	.866	31	3.11	33	.162	36	0	38	0	38	0	38
2		min	-.843	33	-.077	35	-.161	34	0	31	0	31	0	31
3	N1	max	1.019	31	3.162	31	.172	36	0	38	0	38	0	38
4		min	-1.04	33	-.034	37	-.171	38	0	31	0	31	0	31
5	N5	max	1.271	31	6.159	32	.226	36	0	38	0	38	0	38
6		min	-.883	37	-.173	38	-2.349	34	0	31	0	31	0	31
7	N6	max	1.01	35	5.927	32	.214	36	0	38	0	38	0	38
8		min	-1.401	33	-.034	38	-2.339	34	0	31	0	31	0	31
9	N9	max	1.27	31	6.19	34	2.343	32	0	38	0	38	0	38
10		min	-.882	37	-.15	36	-.223	38	0	31	0	31	0	31
11	N10	max	1.009	35	5.957	34	2.333	32	0	38	0	38	0	38
12		min	-1.401	33	-.01	36	-.211	38	0	31	0	31	0	31
13	N13	max	1.017	31	3.148	31	.173	32	0	38	0	38	0	38
14		min	-1.038	33	-.037	37	-.171	38	0	31	0	31	0	31
15	N14	max	.864	31	3.095	33	.163	32	0	38	0	38	0	38
16		min	-.841	33	-.08	35	-.16	38	0	31	0	31	0	31
17	N69	max	.057	35	.372	33	.042	36	0	38	0	38	0	38
18		min	-.057	37	.278	35	-.042	38	0	31	0	31	0	31
19	N70	max	.057	35	.372	33	.042	36	0	38	0	38	0	38
20		min	-.057	37	.278	35	-.042	38	0	31	0	31	0	31
21	N73	max	.057	35	.372	31	.042	36	0	38	0	38	0	38
22		min	-.057	37	.278	37	-.042	38	0	31	0	31	0	31
23	N74	max	.057	35	.372	31	.042	36	0	38	0	38	0	38
24		min	-.057	37	.278	37	-.042	38	0	31	0	31	0	31
25	Totals:	max	8.256	35	24.248	34	5.944	36						
26		min	-8.256	33	14.423	35	-5.944	34						

Use

Company:	SMG	Page:	1
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	10/31/2019
E-Mail:			

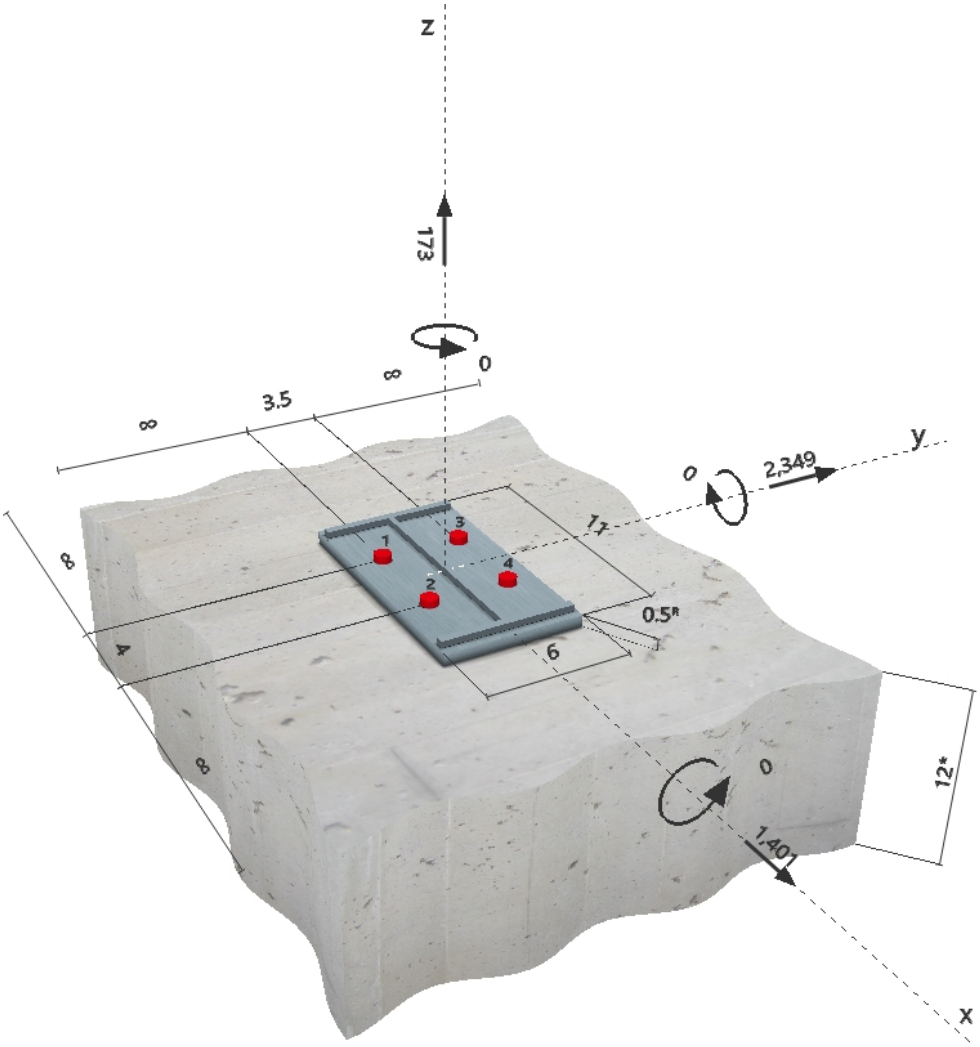
Specifier's comments: Enclose Scale and Loadout

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 11.000 in. x 6.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC), W10X22; (L x W x T x FT) = 10.200 in. x 5.750 in. x 0.240 in. x 0.360 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

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

Geometry [in.] & Loading [lb, in.lb]



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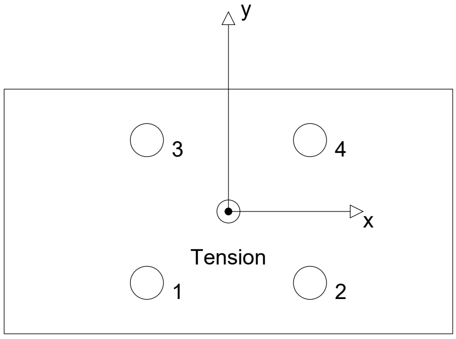
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	10/31/2019
E-Mail:			

2 Load case/Resulting anchor forces

Load case: Design loads



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

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	43	684	350	587
2	43	684	350	587
3	43	684	350	587
4	43	684	350	587

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 173 [lb]
resulting compression force in (x/y)=(0.000/0.000): 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*	43	14,529	1	OK
Pullout Strength*	43	11,479	1	OK
Concrete Breakout Strength**	173	14,807	2	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\ f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [lb]
19,372

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
19,372	0.750	14,529	43

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\ N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8\ A_{brg}\ f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	3,000

Calculations

N_p [lb]
21,864

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
21,864	0.700	0.750	1.000	11,479	43

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Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	10/31/2019
E-Mail:			

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-14 Eq. (17.4.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	∞	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 [lb]
473.00	324.00	1.000	1.000	1.000	1.000	19,320

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
28,204	0.700	0.750	1.000	14,807	173

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Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	10/21/2019
E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	684	7,555	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,735	39,486	7	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading   **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [lb]
11,623

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\, V_{sa}$ [lb]	V_{ua} [lb]
11,623	0.650	7,555	684

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 318-14 Eq. (17.5.3.1b)

$\phi\, V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a\, \sqrt{f_c'}\, h_{ef}^{1.5}$ 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.]
2	6.000	0.000	0.000	∞

$\psi_{c,N}$	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 [lb]
473.00	324.00	1.000	1.000	1.000	1.000	19,320

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, V_{cpg}$ [lb]	V_{ua} [lb]
56,409	0.700	1.000	1.000	39,486	2,735

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.012	0.091	5/3	2	OK

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

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (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 Anchor 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 Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

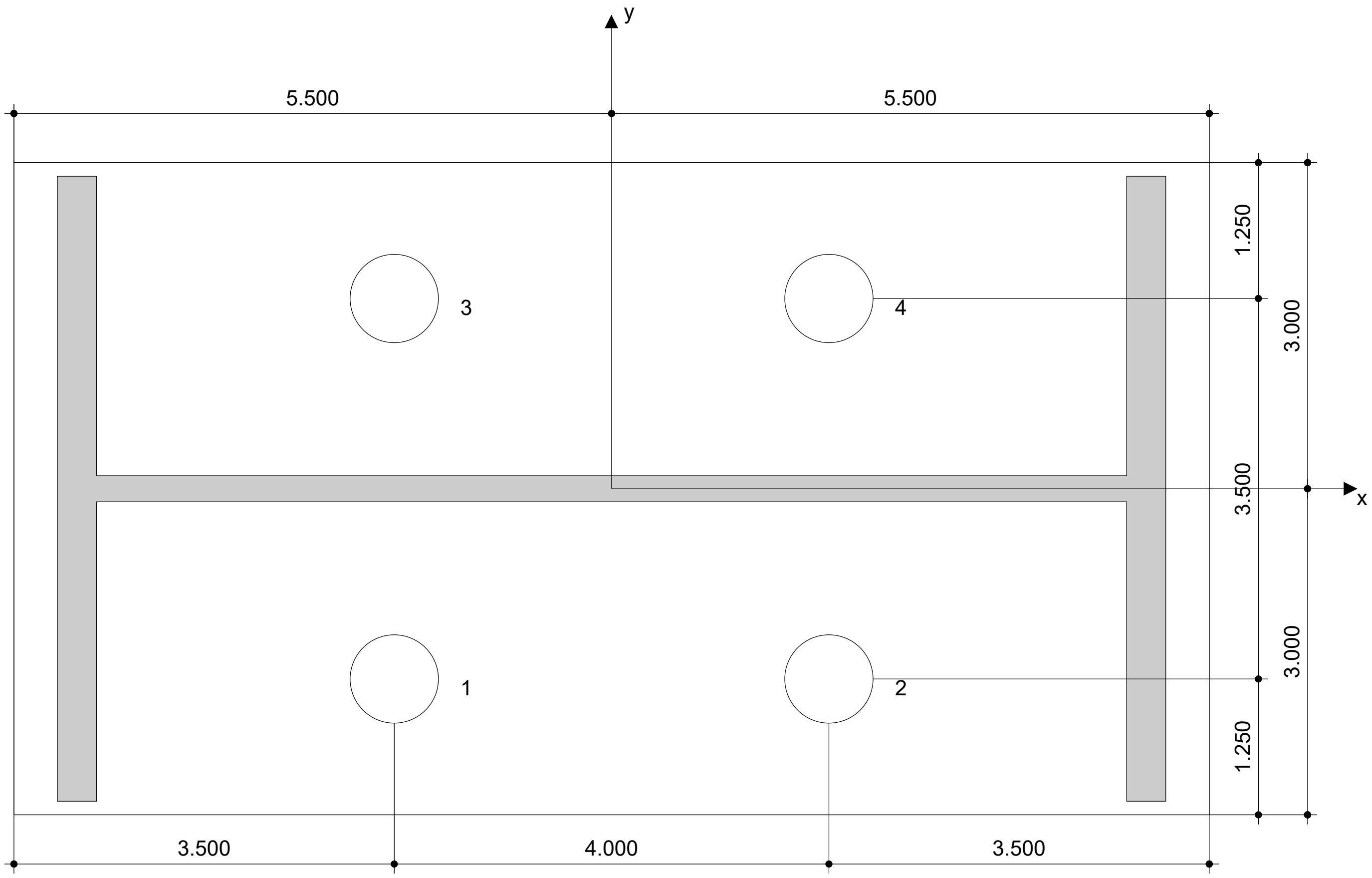
Fastening meets the design criteria!

7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC), W10X22; (L x W x T x FT) = 10.200 in. x 5.750 in. x 0.240 in. x 0.360 in.
Hole diameter in the fixture: d_f = 0.813 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4
Installation torque: -

Hole diameter in the base material: - in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.000 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.000	-1.750	-	-	-	-
2	2.000	-1.750	-	-	-	-
3	-2.000	1.750	-	-	-	-
4	2.000	1.750	-	-	-	-

Smith Monroe Gray
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New Loadout Enclosure - Appended Calcs
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AISC 341-10 Requirements for a OMF

Analysis - no additional requirements
System - no additional requirements
Basic - no limitation on width to thickness beyond what is in AISC360
no requirements for stability bracing of beams or joint beyond AISC360
Protected Zones - no designated protected zones
Connection - can be fully restrained or partially restrained

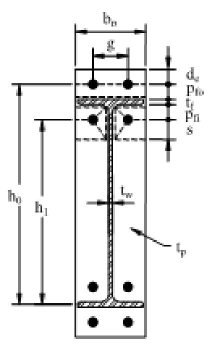
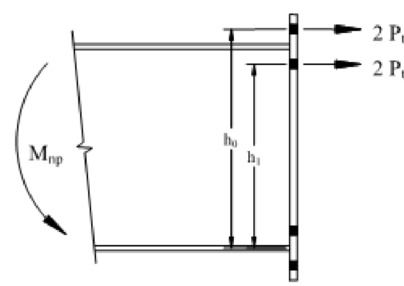
$R_y := 1.1$ $\qquad M_p := 50\ ksi \cdot 31.3\ in^3$ $\qquad L_{cf} := 19\ ft$

$$E_{mh} := 2 \cdot \frac{(1.1 \cdot R_y \cdot M_p)}{L_{cf}} = 16.6\ kip$$

$M_u := (1.1 \cdot R_y \cdot M_p) = 1894\ kip \cdot in$ $\qquad M_u = 157.8\ kip \cdot ft$

Run seismic load combinations without seismic to develop additional shear to add to Emh

$$V_u := 2.1\ kip + E_{mh} = 18.7\ kip$$

End Plate Geometry and Yield Line Pattern	Bolt Force Model
	
End Plate	$\phi M_{pl} = \phi_b F_{yp} t_p^2 Y_p$ $Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g} h_1 (p_{fi} + s)$ $s = \frac{1}{2}\sqrt{b_p g}$ $\quad \phi_b = 0.90$ $\quad$ Note: If $p_{fi} > s$, use $p_{fi} = s$
Bolt Rupture	$\phi M_{np} = \phi\, 2 P_t (h_o + h_1)$ $\quad \phi = 0.75$

$\phi_b := 0.9$ $\qquad t_p := \frac{7}{8} \cdot in$ $\qquad b_p := 5.75\ in$ $\qquad p_{fi} := 1.5\ in$ $\qquad p_{fo} := 1.5\ in$

$F_{yp} := 50\ ksi$ $\qquad g := 2.75\ in$ $\qquad h_1 := 8.36\ in$ $\qquad t_f := 0.44\ in$ $\qquad h_0 := 11.8\ in$

Plate Check

$$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 2\ in$$

$$Y_p := \frac{b_p}{2} \cdot \left(h_1 \cdot \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \cdot \frac{1}{p_{fo}} - \frac{1}{2} \right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(p_{fi} + s \right) \right) = 70.5\ in$$

$$M_{pl} := F_{yp} \cdot {t_p}^2 \cdot Y_p$$

$$\phi_b \cdot M_{pl} = 202.4\ kip \cdot ft$$

Bolt Check

$$bolt_{dia} := \frac{7}{8} \cdot in$$

$$A_b := \frac{{bolt_{dia}}^2 \cdot \pi}{4} \qquad F_t := 90\ ksi$$

$$P_t := F_t \cdot A_b = 54.1\ kip$$

$$2 \cdot P_t \cdot \left(h_0 + h_1 \right) = 182\ kip \cdot ft$$

Check compression bolts shear rupbture strength

$$V_u = 18.7\ kip$$

$$\phi V_n := 4 \cdot 24.3\ kip = 97\ kip$$

Design Welds $d := 10.3\ in$ $b_f := 5.77\ in$ $t_w := 0.26\ in$

$$F_{fu} := \frac{M_u}{d - t_f} = 192.1\ kip$$ Flange force

$$l_e := 2 \cdot b_f - t_w = 11.3\ in$$

$$w_{req} := \frac{F_{fu}}{0.75 \cdot .6 \cdot \frac{70\ ksi}{\sqrt{2}} \cdot 1.5 \cdot l_e} = 0.51\ in$$ Use CJP

$$\phi T := 0.9 \cdot 50\ ksi \cdot t_w = 11.7\ \frac{kip}{in}$$

$$\frac{\phi T}{2 \cdot .75 \cdot .6 \cdot \frac{70\ ksi}{\sqrt{2}} \cdot 1.5} = 0.175\ in$$ Use 3/16, each side

AISC 341-10 Requirements for a OCBF

F5a Satisfy D1.1 — Satisfied as these are tension only rods

F6a

Required Strength $P_u := 3.2\ kip$

Need not exceed
$R_y \cdot F_y \cdot A_g$

OK by inspection

Column Bases

Axial Strength

Required Connection Strengths

Column axial load with amplified siesmic $P_u := 6.2\ kip$

Column shear load with amplified siesmic $1.4\ kip$

Column shear based on column splice (5c)

$$M_{pc} := 50\ ksi \cdot 31.3\ in^3 \qquad H := 17\ ft \qquad V_u := \frac{M_{pc}}{H} = 7.7\ kip$$

Base Plate

$$N := \left(5 + \frac{7}{8}\right) \cdot in \qquad B := 11\ in$$

Check weld

$$3 \cdot 8.25 \cdot 1.392\ kip = 34.5\ kip$$

Weld capacity ok

Smith Monroe Gray
ENGINEERS, INC.

CLIENT Lakeside Industries
PROJECT Maple Valley Asphalt Plant
New Loadout Enclosure - Appended Calcs
BY JWC DATE 11/01/19 REV. 1
JOB NO. 18-183F SHEET 18 OF 22

Base Plates Under Axial Load
Yield Line Approach

Inputs

d = depth of the steel W section	10.3	in
b_f = flange width of the steel W section	5.77	in
P_u = Ultimate Axial Load	6.2	kips
B = Width of Base Plate	11	in
N = Depth of Base Plate	5.875	in
f`c = Bearing Pressure	4	ksi
Fy = Yield Stress of Plate	36	ksi

Outputs

$n` = {}^{1}/_{4}\,(d*b_f)^{0.5} =$ 1.927288

$X = 4 / ((d+b_f)^2\, F_p) * (P_u\, (d*b_f / B*N)) =$ 0.04

$\lambda = 2 * X^{0.5} / 1 + (1-X)^{0.5} =$ 0.20

n = (B - 0.8*d)/2 3.192
m = (N - 0.95 bf)/2 -1.955

$tp = 1.5$ (largest of n, m. or λn`) $(fp/Fy)^{0.5} =$ 0.25 in

Use Baseplate thickness = $\boxed{\textbf{1/2}}$ in

5/8 plate used, OK

Company:	SMG	Page:	1
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	11/9/2019
E-Mail:			

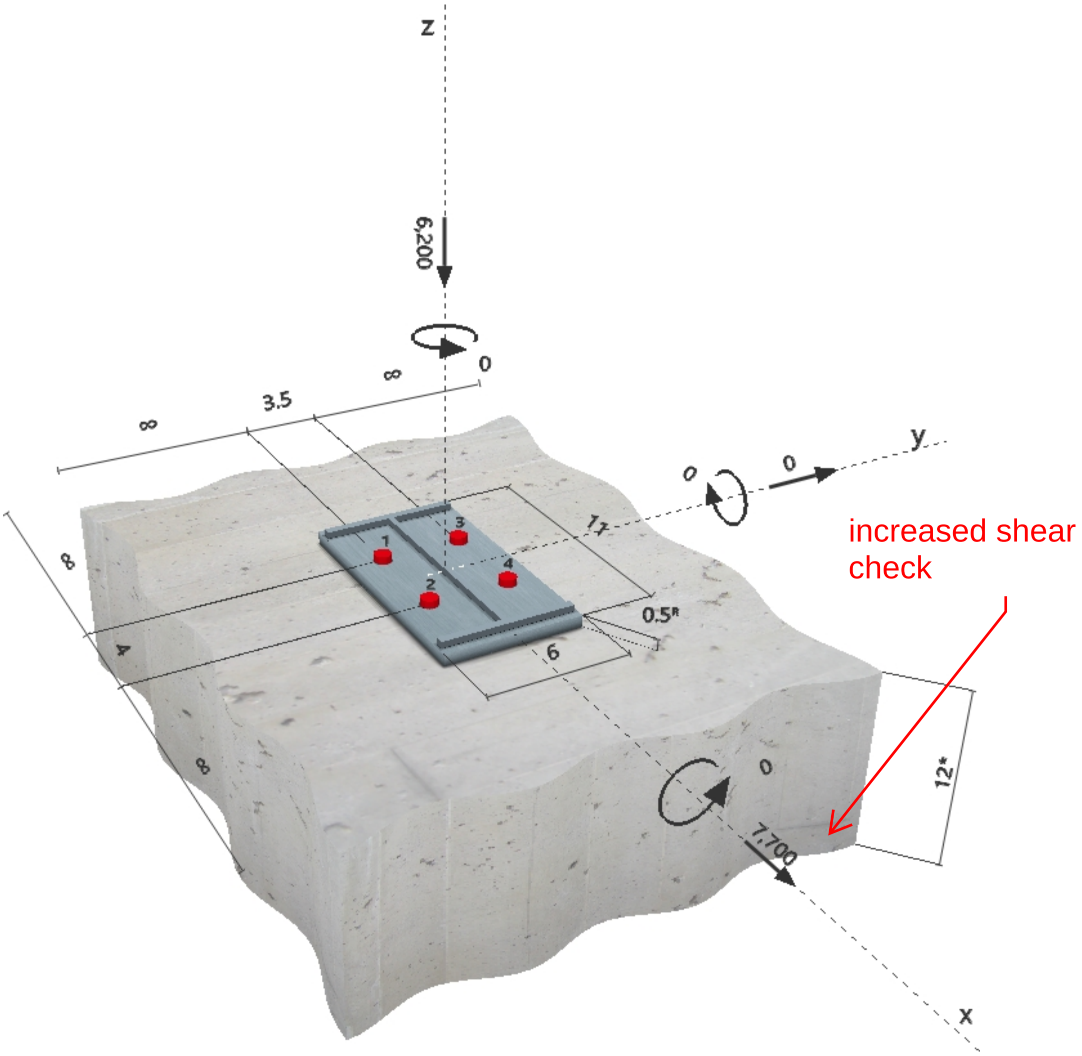
Specifier's comments: Enclose Scale and Loadout - Per Column Bases Requirements

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 11.000 in. x 6.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC), W10X22; (L x W x T x FT) = 10.200 in. x 5.750 in. x 0.240 in. x 0.360 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (a))
	Shear load: yes (17.2.3.5.3 (a))

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

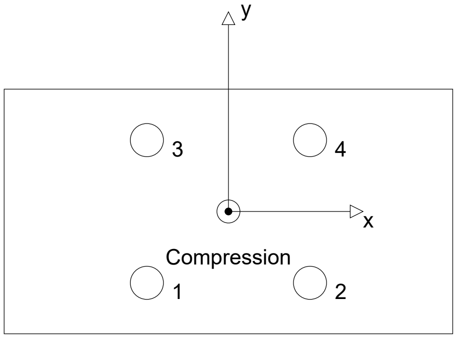
Geometry [in.] & Loading [lb, in.lb]



Company:	SMG	Page:	2
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	11/1/2019 20
E-Mail:			

2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,925	1,925	0
2	0	1,925	1,925	0
3	0	1,925	1,925	0
4	0	1,925	1,925	0

max. concrete compressive strain: 0.02 [‰]
max. concrete compressive stress: 94 [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 6,200 [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 Strength**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading   **anchor group (anchors in tension)

Company:	SMG	Page:	3
Specifier:	JWC	Project:	Lakeside Industries
Address:		Sub-Project I Pos. No.:	Maple Valley
Phone I Fax:	|	Date:	12/1/2019
E-Mail:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,925	7,555	26	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	7,700	39,486	20	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [lb]
11,623

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
11,623	0.650	7,555	1,925

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 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ 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.]
2	6.000	0.000	0.000	∞

$\psi_{c,N}$	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 [lb]
473.00	324.00	1.000	1.000	1.000	1.000	19,320

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
56,409	0.700	1.000	1.000	39,486	7,700

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (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 Anchor 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 Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!