Eastern Engineering Group

3401 NW 82nd Avenue Suite 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

STRUCTURAL CALCULATIONS

Project name:

10635 SW 63rd AVE

Job No.:

26-0297

Project address:

10635 SW 63rd Ave, Miami, FL 33156, USA

These calculations are strictly valid for the address above. Use in other projects is unauthorized and may constitute fraud.

Submission No :	SUBMITTAL REVIEW Z. BELFRANIN PE		
Project No : 10635 S.W. 63rd Avenue		Discipline :	
X	Reviewed– No Exceptions Taken		Reviewed–Revise and Resubmit
	Reviewed–As Noted		Reviewed by Consultant Not Required

Review is solely for general conformity with contract. The consultant does not warrant or represent that information in this submittal is accurate or complete. Review does not relieve Spatiality Engineer/Contractor of responsibility for error/omissions in design, including this submittal, that are the Spatiality Engineer/ Contractor's responsibility, and for conforming/correlating with all quantities/dimensions, performing the Work, selecting performance means/methods, coordinating with other parts of the Work/between trades, and performing the Work safety. Notwithstanding this review, Spatiality Engineer/ Contractor remains solely responsible for contract compliance.

By : ZTB — Date APRIL 30, 2026

GONZALO A. PAZ
LICENSE
No 60734
★
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

Digitally signed by Gonzalo A Paz
Date: 2026.04.30 15:37:22-04'00'

Prepared by: Gonzalo Paz, PE
Lic. No. 60734/CAN # 26655

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronics copies

THIS ITEM HAS BEEN DIGITALLY SIGNED AND SEALED BY **GONZALO A. PAZ, PE** ON THE DATE ADJACENT TO THE SEAL.

PRINTED COPIES OF THIS DOCUMENT ARE NOT CONSIDERED SIGNED AND SEALED AND THE SIGNATURE MUST BE VERIFIED ON ANY ELECTRONIC COPIES.

Eastern Engineering Group

3401 NW 82nd Avenue Suite 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

DESIGN CRITERIA:

Calculations based on:

1. Florida Building Code 8th Edition 2023
2. Minimum Design Loads for Buildings and Other Structures ASCE 7-22
3. Building Code Requirements for Structural Concrete ACI 318-19
4. American Institute of Steel Construction AISC 360-16
5. Aluminum Design Manual 2020
6. American Wood Council NDS 2024

CALCULATION INDEX:

Total Pages = 109

CALCULATION STATEMENT:

To the best of my knowledge, ability, belief and professional judgment, I hereby attest that the manual calculations and computer-generated calculations are in compliance with the existing governing codes.

Submission No :	SUBMITTAL REVIEW Z. BELFRANIN PE		
Project No : 10635 S.W. 63rd Avenue		Discipline :	
X	Reviewed– No Exceptions Taken		Reviewed–Revise and Resubmit
	Reviewed–As Noted		Reviewed by Consultant Not Required

Review is solely for general conformity with contract. The consultant does not warrant or represent that information in this submittal is accurate or complete. Review does not relieve Spatiality Engineer/Contractor of responsibility for error/omissions in design, including this submittal, that are the Spatiality Engineer/ Contractor's responsibility, and for conforming/correlating with all quantities/dimensions, performing the Work, selecting performance means/methods, coordinating with other parts of the Work/between trades, and performing the Work safety. Notwithstanding this review, Spatiality Engineer/ Contractor remains solely responsible for contract compliance.

By : ZTB | Date APRIL 30, 2026

GONZALO A. PAZ
LICENSE
No 60734
STATE OF FLORIDA
PROFESSIONAL ENGINEER

Digitally signed by Gonzalo A Paz
Date: 2026.04.30 15:37:12-04'00'

Prepared by: Gonzalo Paz, PE
Lic. No. 60734/CAN # 26655

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronics copies

THIS ITEM HAS BEEN DIGITALLY SIGNED AND SEALED BY **GONZALO A. PAZ, PE** ON THE DATE ADJACENT TO THE SEAL.

PRINTED COPIES OF THIS DOCUMENT ARE NOT CONSIDERED SIGNED AND SEALED AND THE SIGNATURE MUST BE VERIFIED ON ANY ELECTRONIC COPIES.

Eastern
Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

WIND ANALYSIS

Submission No :	SUBMITTAL REVIEW Z. BELFRANIN PE		
Project No : 10635 S.W. 63rd Avenue		Discipline :	
X	Reviewed– No Exceptions Taken		Reviewed–Revise and Resubmit
	Reviewed–As Noted		Reviewed by Consultant Not Required

Review is solely for general conformity with contract. The consultant does not warrant or represent that information in this submittal is accurate or complete. Review does not relieve Spatiality Engineer/Contractor of responsibility for error/omissions in design, including this submittal, that are the Spatiality Engineer/ Contractor's responsibility, and for conforming/correlating with all quantities/dimensions, performing the Work, selecting performance means/methods, coordinating with other parts of the Work/between trades, and performing the Work safety. Notwithstanding this review, Spatiality Engineer/ Contractor remains solely responsible for contract compliance.

By : ZTB   Date APRIL 30, 2026

Wind for Solid Freestanding Walls & Signs Design (ε>70%) ASCE 7-22

General Wind Data:

Miami Dade
Category II.............V=175 mph

$V := 175.00$ Wind Velocity (mph)

$K_{zt} := 1.00$ Topographic Factor

$K_d := 0.85$ Wind Directionality Factor (see table 26.6-1) ASCE 7-22 ....**Kd=0.85**

$G := 0.85$ Gust Factor

$C_f := 1.80$ Net Force Coefficients (Figure 29.3-1)

For Solid Signs: s/h<0.16 & 0.2<B/s<10......Cf=1.85
For Freestanding Walls: s/h>=1 & B/s=1...............Cf=1.45
s/h>=1 & B/s=2..................Cf=1.40
s/h>=1 & B/s=5..................Cf=1.35
s/h>=1 & B/s=10................Cf=1.30

$\alpha := 9.8$

$zg := 2460.0$

Values for Terrain exposure constants α and zg:
Exposure B------ Value α =7.5 , Value zg=3,280
Exposure C------ Value α =9.8 , Value zg=2,460
Exposure D------ Value α =11.5 , Value zg=1,935

General Sign Data:

$Z := 18.33$ Height of Top of Sign (ft)

$\varepsilon := 100.0$ Solidity Ratio of Sign (%)

$$\lambda := 1 - \left(1 - \frac{\varepsilon}{100}\right)^{1.5}$$

$\lambda = 1.00$ Solidity Ratio of Sign (%)

Then

$$Z := \text{if}(Z < 15, 15, Z)$$

$$K_z := 2.41\left(\frac{Z}{zg}\right)^{\frac{2}{\alpha}}$$

$K_z = 0.89$

$$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$$

$q_z = 59.09$ psf

Allowable Design Wind Loads:

$$p_z := 0.6\left(q_z \cdot G \cdot C_f\right)$$

$p_z = 54.24$ Gross Wind per Actual Solid Area (psf)

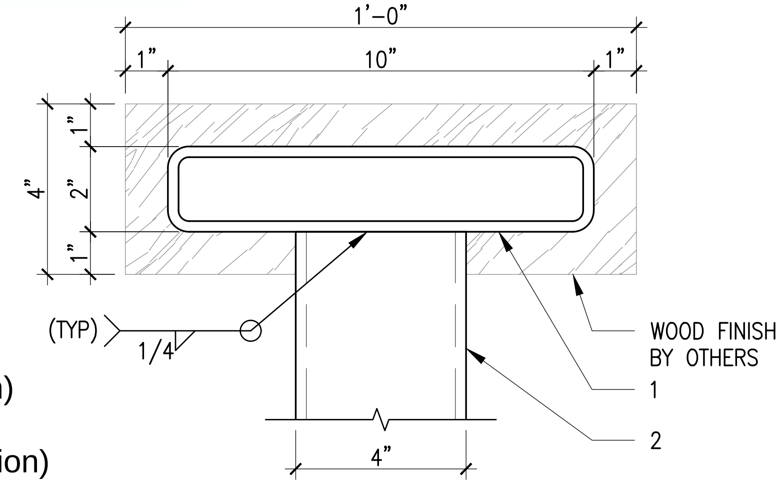
$$p_{z.EQ} := \max\left(\lambda \cdot p_z, 10\right)$$

$p_{z.EQ} = 54.24$ Equivalent Wind in Overall Area (psf)

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

STEEL STAIR DESIGN

TREAD DESIGN



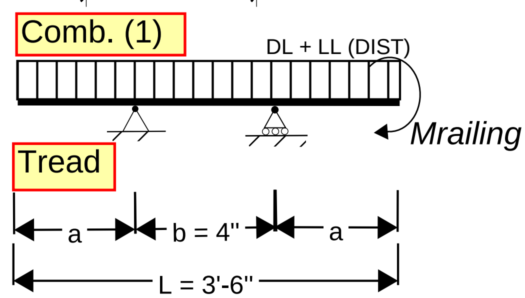
Loads:

Mrailing = 650#-ft (due to 200# railing)
DL= DLsystem +DL Super. = 35 psf
LL = 100 psf (dist.) →(Conserv.)
PLL= 300# (conc.)

Loads Combinations:

Comb (1): DL + LL + (Mrailing or 200# railing action)

Comb (2): DL + 300# + (Mrailing or 200# railing action)

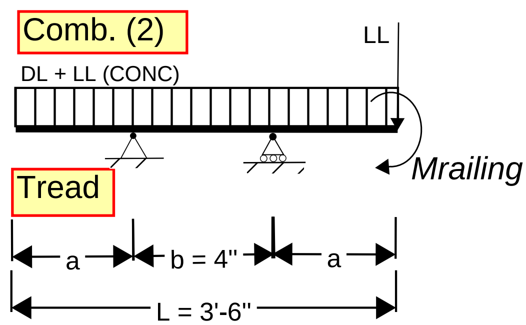


Loads Processing:

Tributary Strip=12"/12 = 1.0 ft
La = 1.58 ft
Lb = 0.33 ft

For Tread Analysis see spreadsheet printout.
Combination 2 is the worst case scenario.

Comb (1): DL + LL + Mrailing
M1= 0.819 k-ft



Comb (2): DL + 300#
M2= 1.168 k-ft = 14016 #-in **(WORST CASE)**

fb= M/Sx ≤Fb= 27 600 psi
fb= 14016 #-in/3.8495 in^3
fb= 3640.99.00 psi < 27 600 psi (OK)

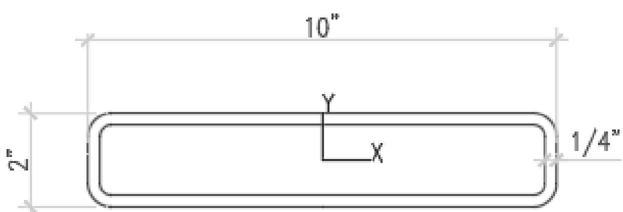
Local Buckling Analysis:
M2 = 14016 #-in

P = M2/d = 14016 #-in/4" = 3504 #

Mactual = P*L/4
Mactual = 3504# * 10"/4
Mactual = 8760 #-in < Mallow = 106 246 #-in (OK)

Area = 5.5890 in^2
Ixx = 3.8495 in^4
Iyy = 55.5472 in^4
Jz = 59.3967 in^4
Pxy = 0.0000 in^4
rxx = 0.8299 in
ryy = 3.1526 in
Sxx max = 3.8495 in^3
Sxx min = 3.8495 in^3
Syy max = 11.1094 in^3
Syy min = 11.1094 in^3

HSS10X2X1/4
STEEL TREAD



Steel Beam

Project File: 26-0297.ec6

LIC# : KW-06018068, Build:20.25.07.09 Eastern Engineering Group (c) ENERCALC, LLC 1982-2026

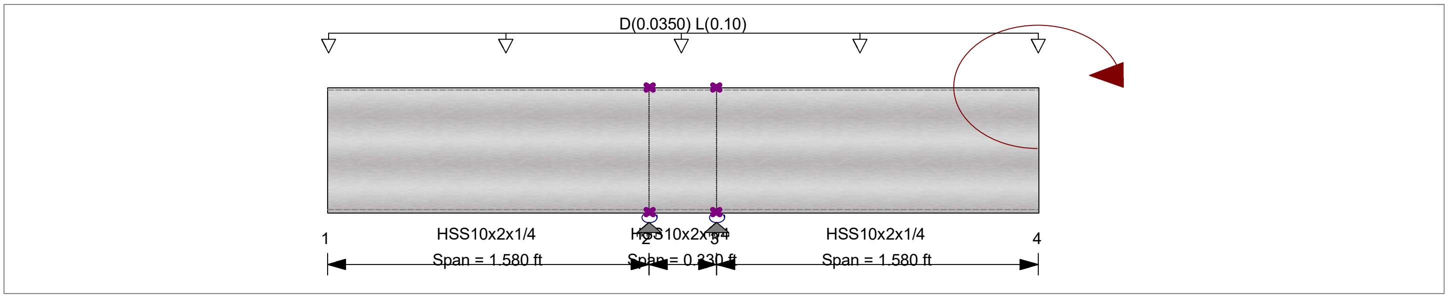
DESCRIPTION: Combination #1 (DL+LL+Mrailing or 200#)

Code References

Governing Code : IBC 2018, CBC 2019
Referenced Design Standard(s) : AISC 360-22* *User-selected, not referenced by : IBC 2018, CBC 2019
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0350, L = 0.10 k/ft
Load(s) for Span Number 3
Moment : L = 0.650 k-ft, Loc = 1.580 ft in span

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.105** : 1	Maximum Shear Stress Ratio =	**0.199** : 1
Section used for this span	**HSS10x2x1/4**	Section used for this span	**HSS10x2x1/4**
Ma : Applied	0.819 k-ft	Va : Applied	1.992 k
Mn / Omega : Allowable	7.792 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.330 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.017 in	Ratio =	2,270 >=360	Span: 3 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.017 in	Ratio =	2188 >=360.	Span: 3 : +D+L
Max Upward Total Deflection	-0.000 in	Ratio =	35627 >=360.	Span: 2 : +D+L

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length		Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mny	Mny/Omega	Cb	Rm	Summary of Shear Values Va Max	Vny	Vny/Omega
D Only														
Dsgn. L =	1.58 ft	1	0.006	0.006		-0.04	0.04	13.01	7.79	1.00	1.00	0.06	16.73	10.02
Dsgn. L =	0.33 ft	2	0.006	0.001	-0.00	-0.04	0.04	13.01	7.79	1.01	1.00	0.01	16.73	10.02
Dsgn. L =	1.58 ft	3	0.006	0.006		-0.04	0.04	13.01	7.79	1.00	1.00	0.06	16.73	10.02
+D+L														
Dsgn. L =	1.58 ft	1	0.022	0.021		-0.17	0.17	13.01	7.79	1.00	1.00	0.21	16.73	10.02
Dsgn. L =	0.33 ft	2	0.105	0.199	-0.00	-0.82	0.82	13.01	7.79	1.47	1.00	1.99	16.73	10.02
Dsgn. L =	1.58 ft	3	0.105	0.021		-0.82	0.82	13.01	7.79	1.00	1.00	0.21	16.73	10.02
+D+0.750L														
Dsgn. L =	1.58 ft	1	0.018	0.017		-0.14	0.14	13.01	7.79	1.00	1.00	0.17	16.73	10.02
Dsgn. L =	0.33 ft	2	0.080	0.149	-0.00	-0.62	0.62	13.01	7.79	1.46	1.00	1.50	16.73	10.02
Dsgn. L =	1.58 ft	3	0.080	0.017		-0.62	0.62	13.01	7.79	1.00	1.00	0.17	16.73	10.02
+0.60D														
Dsgn. L =	1.58 ft	1	0.003	0.003		-0.03	0.03	13.01	7.79	1.00	1.00	0.03	16.73	10.02
Dsgn. L =	0.33 ft	2	0.003	0.000	-0.00	-0.03	0.03	13.01	7.79	1.01	1.00	0.00	16.73	10.02
Dsgn. L =	1.58 ft	3	0.003	0.003		-0.03	0.03	13.01	7.79	1.00	1.00	0.03	16.73	10.02

Steel Beam

Project File: 26-0297.ec6

LIC# : KW-06018068, Build:20.25.07.09 | Eastern Engineering Group | (c) ENERCALC, LLC 1982-2026

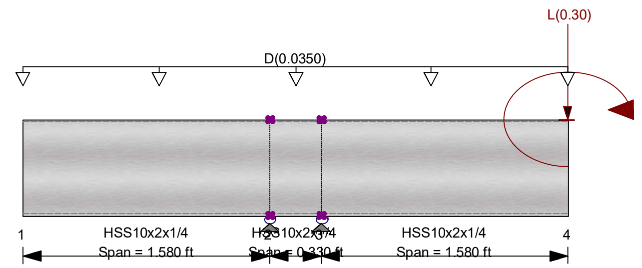
DESCRIPTION: Combination #2 (DL (Dist.)+300#+Mrailing or 200#)

Code References

Governing Code : IBC 2018, CBC 2019
Referenced Design Standard(s) : AISC 360-22* *User-selected, not referenced by : IBC 2018, CBC 2019
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0350 k/ft
Load(s) for Span Number 3
Point Load : L = 0.30 k @ 1.580 ft
Moment : L = 0.650 k-ft, Loc = 1.580 ft in span

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.150** : 1	Maximum Shear Stress Ratio =	**0.341** : 1
Section used for this span	**HSS10x2x1/4**	Section used for this span	**HSS10x2x1/4**
Ma : Applied	1.168 k-ft	Va : Applied	3.412 k
Mn / Omega : Allowable	7.792 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.330 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.023 in	Ratio =	1,675	>=360	Span: 3 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.023 in	Ratio =	1631	>=360.	Span: 3 : +D+L
Max Upward Total Deflection	-0.000 in	Ratio =	28681	>=360.	Span: 2 : +D+L

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 1.58 ft	1	0.006	0.006										10.02
Dsgn. L = 0.33 ft	2	0.006	0.001	-0.00									10.02
Dsgn. L = 1.58 ft	3	0.006	0.006										10.02
+D+L													
Dsgn. L = 1.58 ft	1	0.006	0.006										10.02
Dsgn. L = 0.33 ft	2	0.150	0.341	-0.00									10.02
Dsgn. L = 1.58 ft	3	0.150	0.035		-1.17	1.17	13.01	7.79	1.00	1.00	0.36	16.73	10.02
+D+0.750L													
Dsgn. L = 1.58 ft	1	0.006	0.006		-0.04	0.04	13.01	7.79	1.00	1.00	0.06	16.73	10.02
Dsgn. L = 0.33 ft	2	0.114	0.256	-0.00	-0.89	0.89	13.01	7.79	1.62	1.00	2.56	16.73	10.02
Dsgn. L = 1.58 ft	3	0.114	0.028		-0.89	0.89	13.01	7.79	1.00	1.00	0.28	16.73	10.02
+0.60D													
Dsgn. L = 1.58 ft	1	0.003	0.003		-0.03	0.03	13.01	7.79	1.00	1.00	0.03	16.73	10.02
Dsgn. L = 0.33 ft	2	0.003	0.000	-0.00	-0.03	0.03	13.01	7.79	1.01	1.00	0.00	16.73	10.02
Dsgn. L = 1.58 ft	3	0.003	0.003		-0.03	0.03	13.01	7.79	1.00	1.00	0.03	16.73	10.02

TREAD DEFLECTION ANALYSIS
Max. Transient Deflection = 0.023 in
Allowable deflection = 2L/360
Allowable deflection = (2*21")/360 = 0.116 in

Eastern Engineering Group
3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
Phone: (305) 599-8133
info@easterneg.com
www.easterneg.com

STRINGER DESIGN (HSS8X4X1/4)

Loads:

DL = 35 psf
LL = 60 psf or 300 Conc.
Mrailing = 7600 #-in

Loads Combinations:

(1) DL+LL (distributed) stringer fully loaded
(2) DL+LL (concentrated) stringer fully loaded
(3) DL+LL stringer partially loaded

Loads Processing:

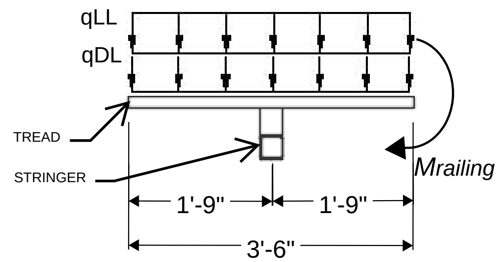
Tributary Strips:
- For DL in all combinations = 3.50ft
- For LL in Comb. 1 = 3.50 ft
- For LL in Comb. 3 = 1.75 ft

qSuperimposed= 35 psf x 3.50 ft = 122.5 #/ft
qLL full= 60 psf x 3.50 ft = 210 #/ft
qLL partial= 60 psf x1.75 ft = 105 #/ft
MLL = 60 psf x 1.75ft x 1.75ft/2 = 92 #-ft/ft

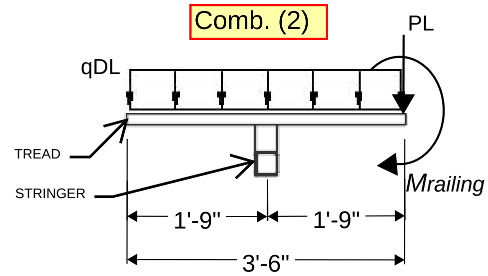
qsuperimposed (Landing) = 35 psf x 3.91ft = 136.85#/ft
qLL (Full) (Landing) = 60 psf x 3.91 ft = 234.6 #/ft
qLL (Partial) (Landing) = 60 psf x 1.95 ft = 117 #/ft
MLL (landing) = 60 psf x 1.95ft x 1.95ft/2 = 114.075 #/ft

(See next Risa calculations)

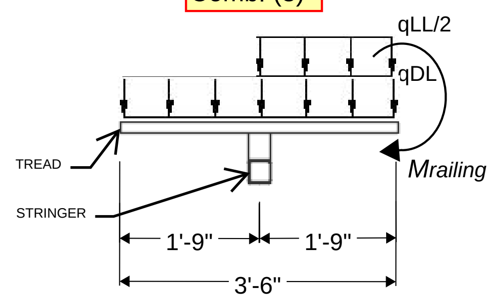
Comb. (1)



Comb. (2)



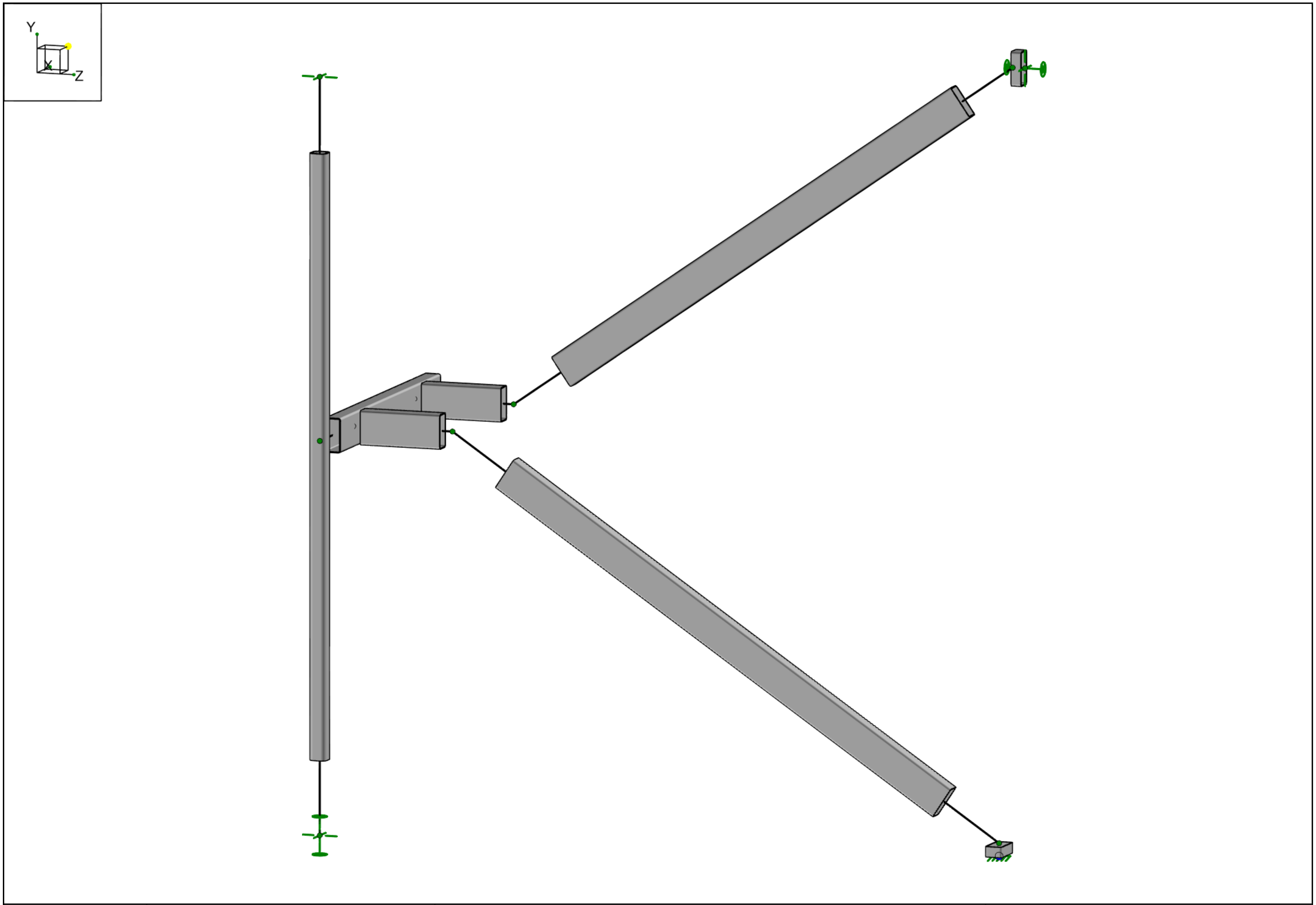
Comb. (3)



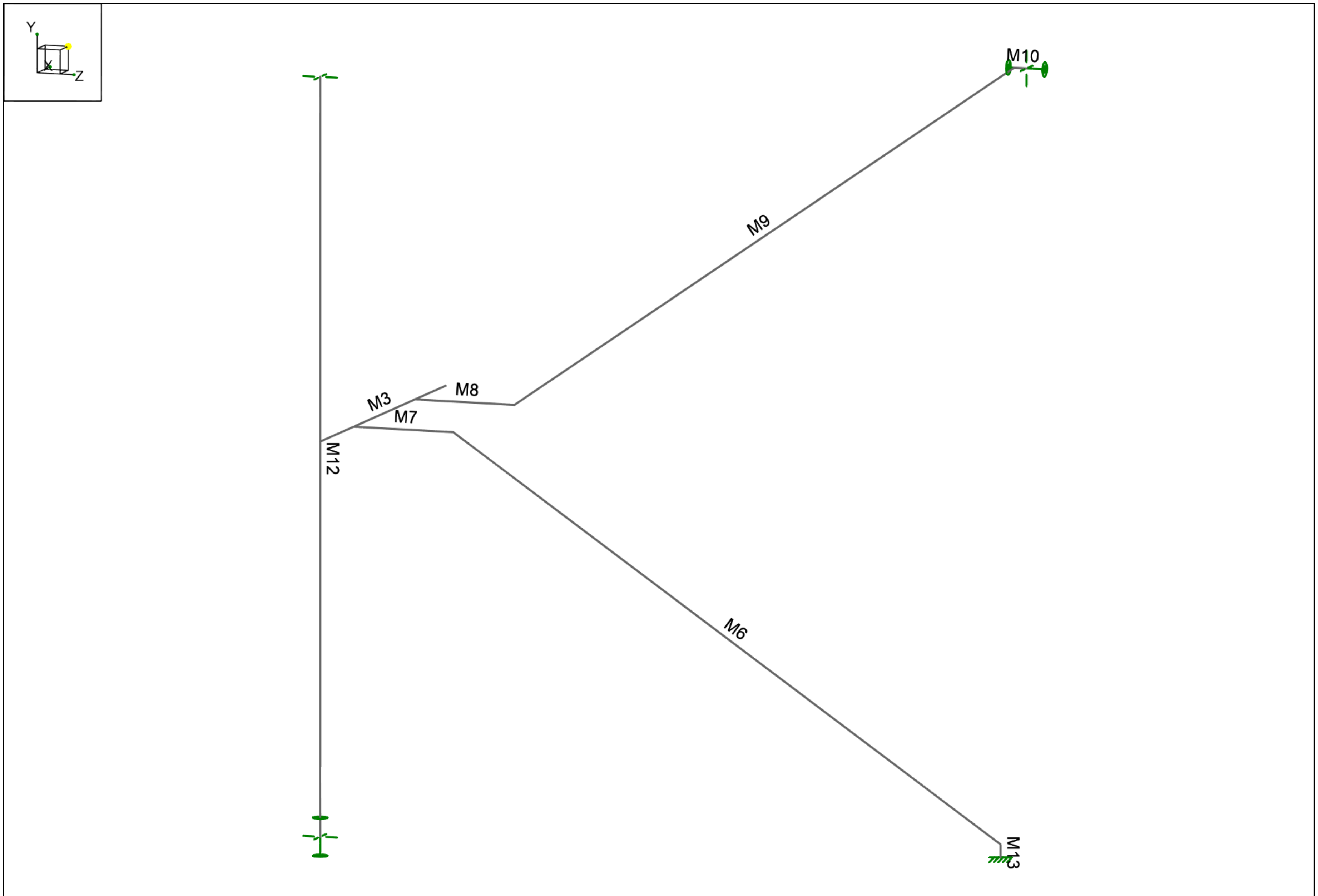
Eastern
Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

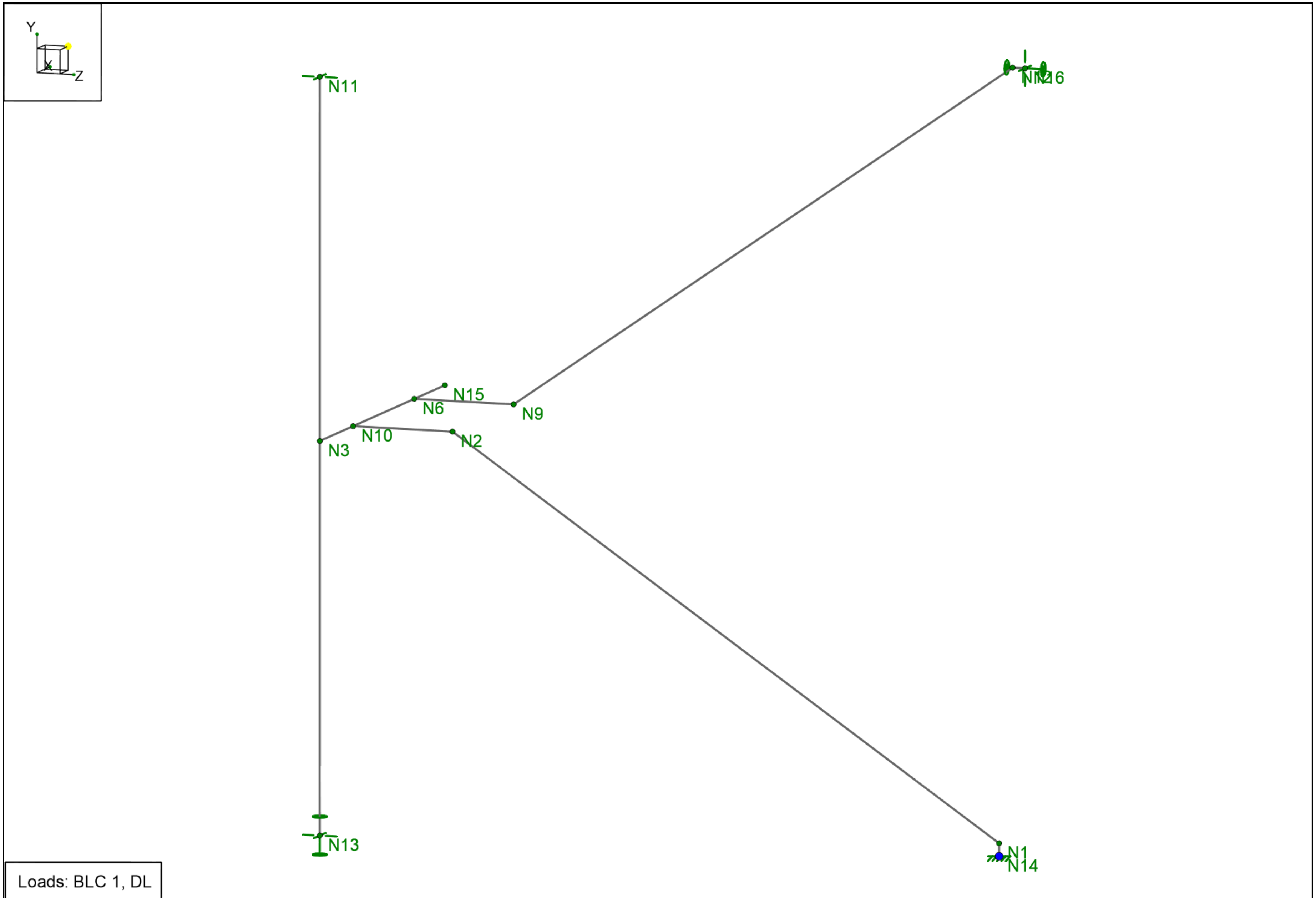
GRAPHICS
Software - Risa-3D
v.22.0.1



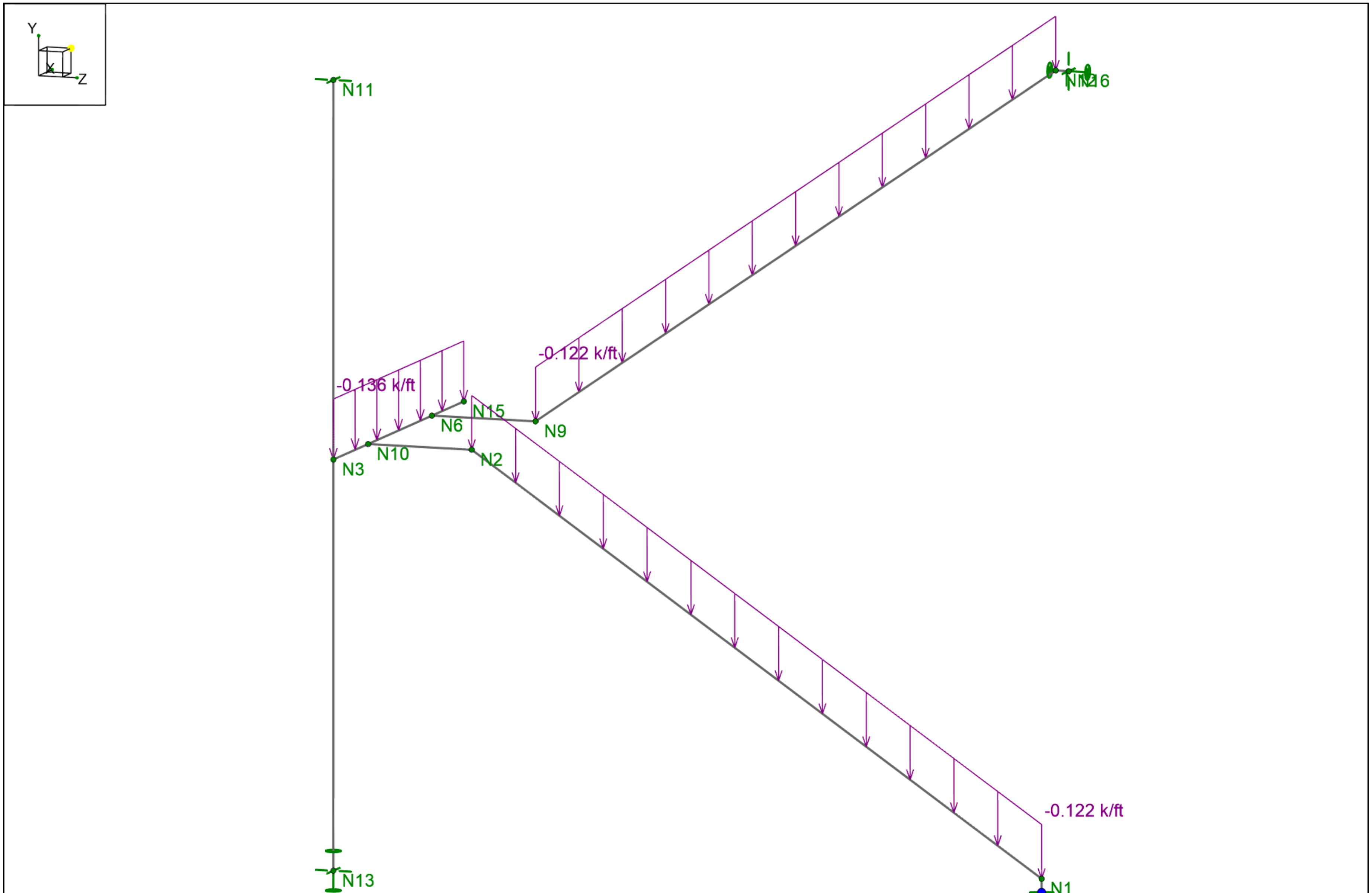
EEG	Stair Design	SK-1
M.E.P		Mar 16, 2026 at 05:34 PM
26-0297		Main Stair Design V03.r3d



RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-2
	M.E.P		Mar 16, 2026 at 05:35 PM
	26-0297		Main Stair Design V03.r3d

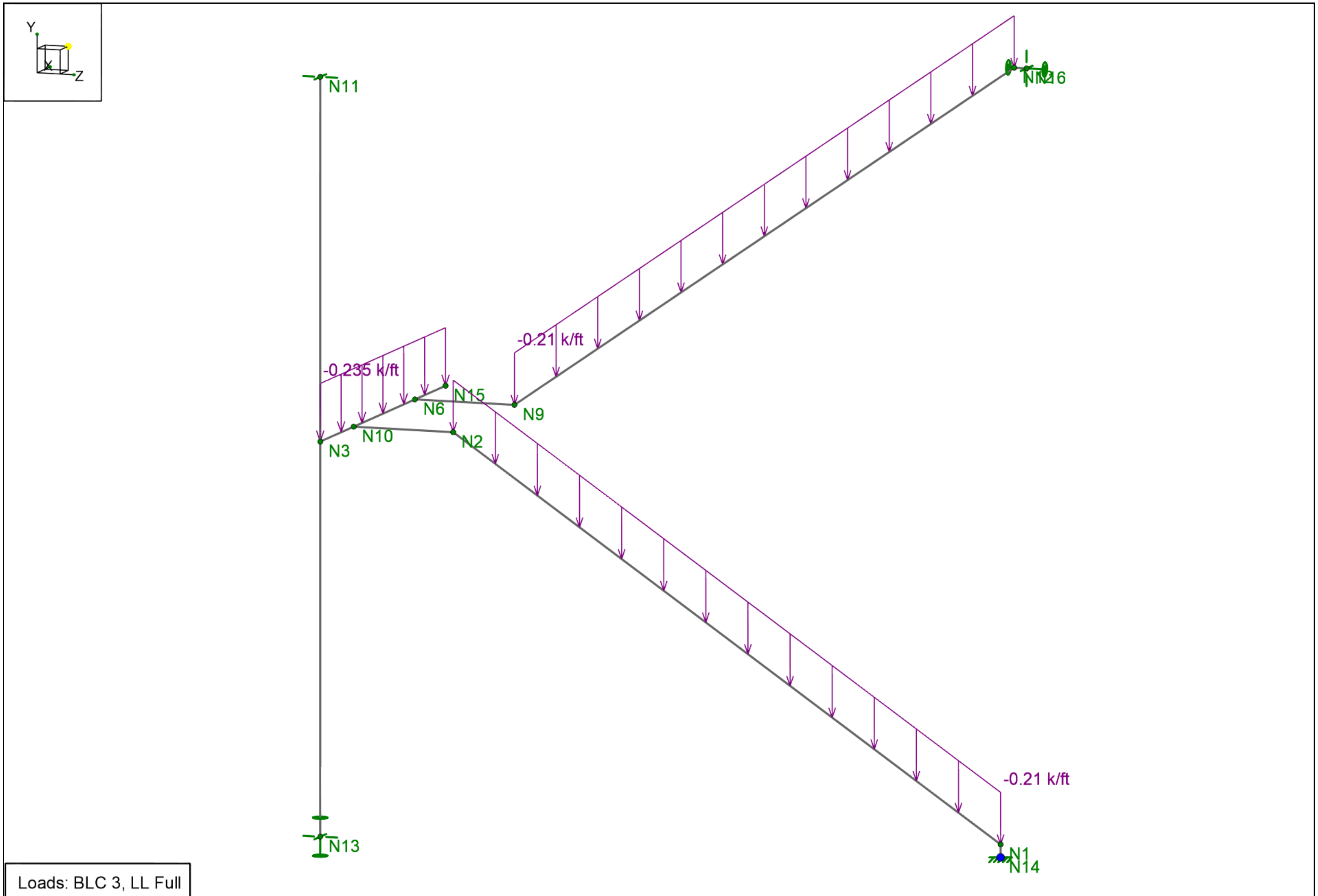


RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-3
	M.E.P		Mar 16, 2026 at 05:35 PM
	26-0297		Main Stair Design V03.r3d



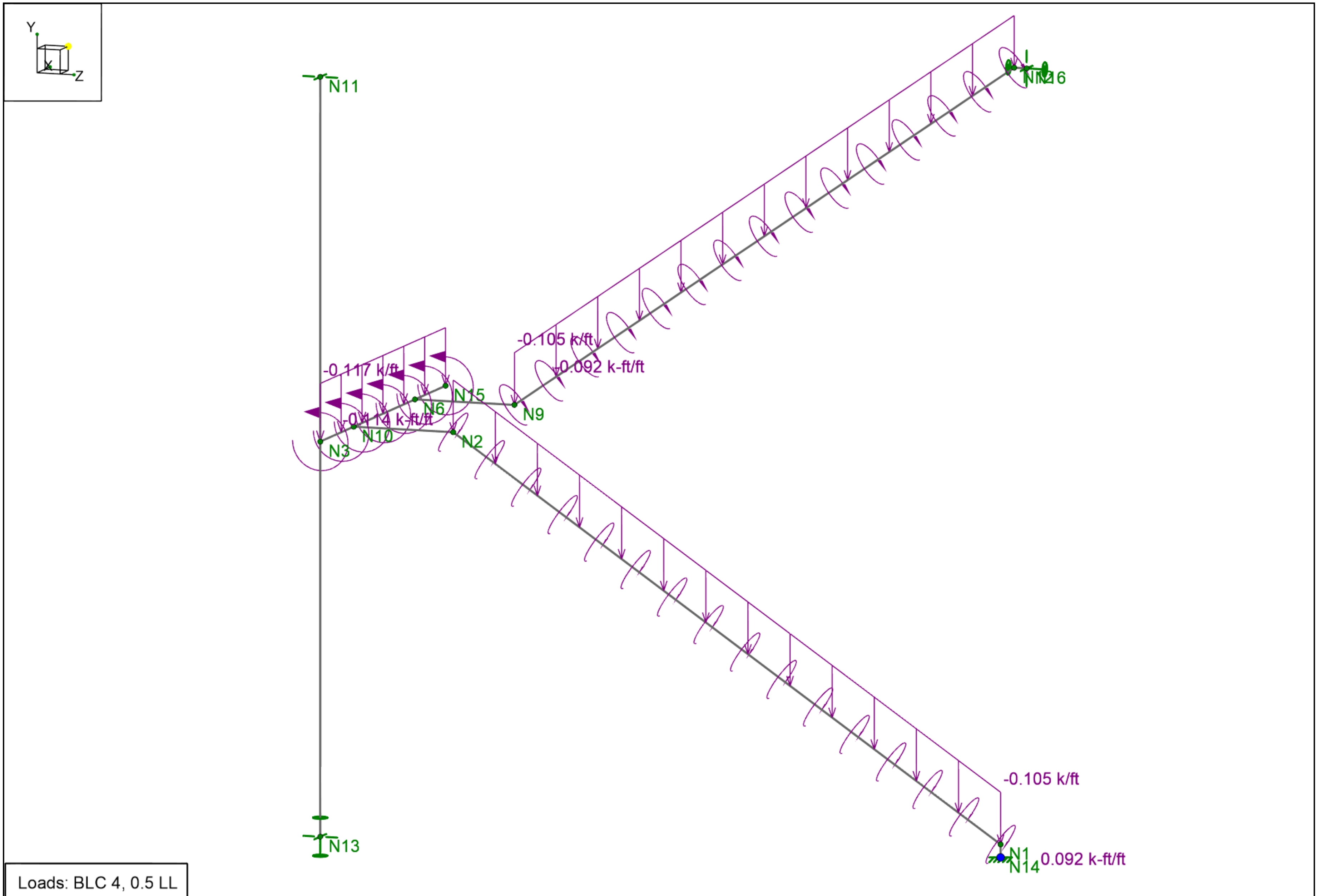
Loads: BLC 2, Super Imposed

RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-4
	M.E.P		Mar 16, 2026 at 05:35 PM
	26-0297		Main Stair Design V03.r3d



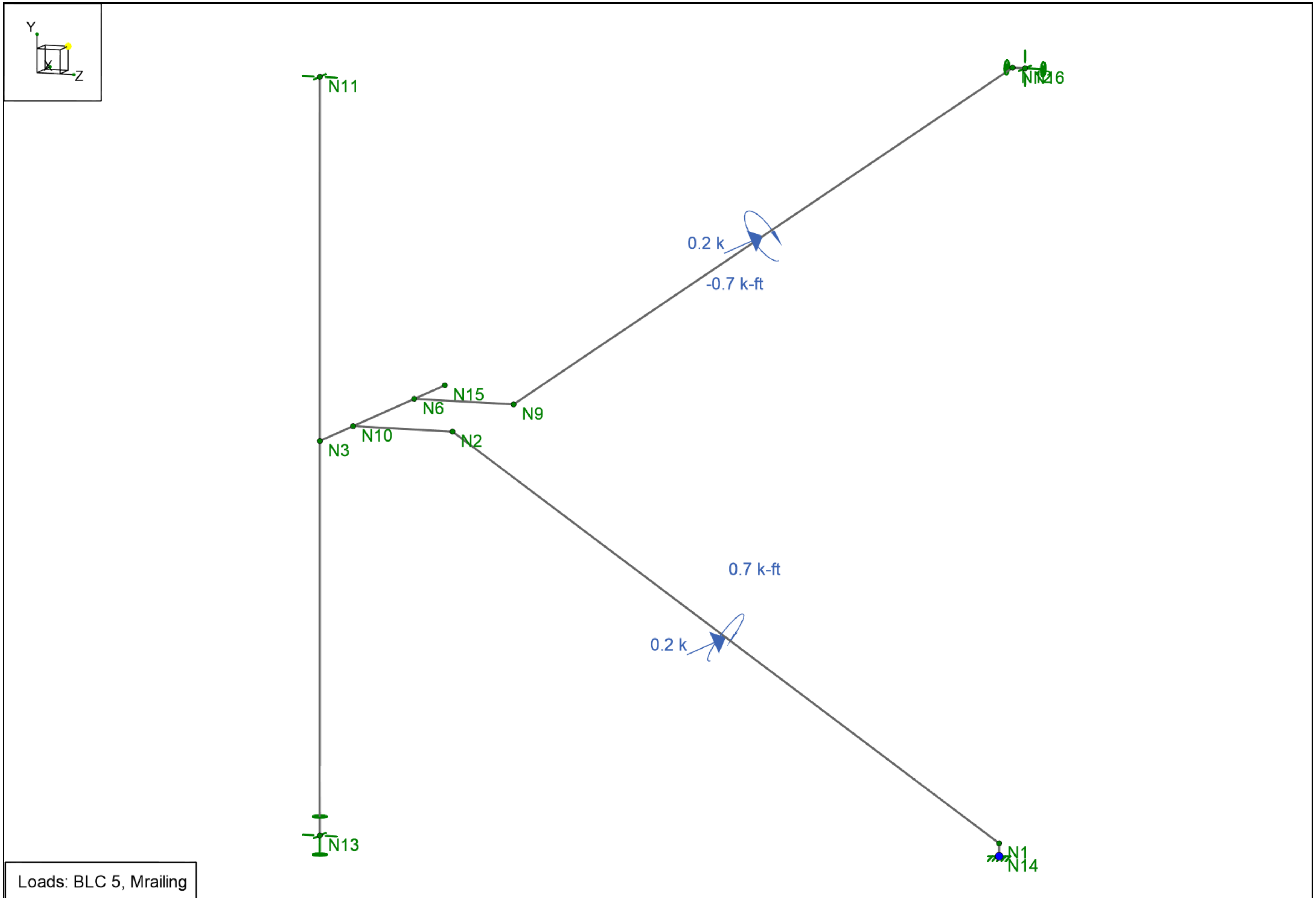
Loads: BLC 3, LL Full

RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-5
	M.E.P		Mar 16, 2026 at 05:35 PM
	26-0297		Main Stair Design V03.r3d



Loads: BLC 4, 0.5 LL

RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-6
	M.E.P		Mar 16, 2026 at 05:36 PM
	26-0297		Main Stair Design V03.r3d



RISA A NEMETSCHEK COMPANY	EEG	Stair Design	SK-7
	M.E.P		Mar 16, 2026 at 05:36 PM
	26-0297		Main Stair Design V03.r3d

Eastern
Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

SOFTWARE: RISA 3D
RISA-3D V.22.0.1
INPUT

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	-0.1165	-0.834	2.25	
2	N2	-0.1165	6.499	-8.75	
3	N3	-2.0265	6.499	-10.75	
4	N6	3.3835	6.499	-10.75	
5	N9	3.3835	6.499	-8.75	
6	N13	-2.0265	-1.0843	-10.75	
7	N14	-0.1165	-1.084	2.25	
8	N12	3.3835	13.499	1.2916	
9	N10	-0.1165	6.499	-10.75	
10	N11	-2.0265	13.499	-10.75	
11	N15	5.1335	6.499	-10.75	
12	N16	3.3835	13.499	1.5416	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N11	Reaction		Reaction			
2	N13	Reaction	Reaction	Reaction		Reaction	
3	N14	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
4	N16	Reaction	Reaction	Reaction			Reaction

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e^{5}°F^{-1}]	Density [k/ft^3]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A992	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	0.3	0.65	0.527	42	1.4	58	1.3
5	A500 Gr.B RECT	29000	11154	0.3	0.65	0.527	46	1.4	58	1.3
6	A500 Gr.C RND	29000	11154	0.3	0.65	0.527	46	1.4	62	1.3
7	A500 Gr.C RECT	29000	11154	0.3	0.65	0.527	50	1.4	62	1.3
8	A53 Gr.B	29000	11154	0.3	0.65	0.49	35	1.6	60	1.2
9	A1085	29000	11154	0.3	0.65	0.49	50	1.4	65	1.3
10	A913 Gr.65	29000	11154	0.3	0.65	0.49	65	1.1	80	1.1

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Main Stringer	HSS8X4X4	Beam	Tube	A500 Gr.B RECT	Typical	5.24	14.4	42.5	35.3
2	Beam	HSS8X4X4	Beam	Tube	A500 Gr.B RECT	Typical	5.24	14.4	42.5	35.3
3	Column	HSS4X4X8	Column	Tube	A500 Gr.B RECT	Typical	6.02	11.9	11.9	21
4	Main Stringer Start	HSS8X4X4	Column	Tube	A500 Gr.B RECT	Typical	5.24	14.4	42.5	35.3

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lcomp top [ft]	Channel Conn.	a [ft]	Function
1	M3	Main Stringer	7.16	Lbyy	N/A	N/A	Lateral
2	M6	Main Stringer	13.22	Lbyy	N/A	N/A	Lateral
3	M7	Main Stringer	2	Lbyy	N/A	N/A	Lateral
4	M8	Main Stringer	2	Lbyy	N/A	N/A	Lateral
5	M9	Main Stringer	12.241	Lbyy	N/A	N/A	Lateral
6	M12	Column	14.583	Lbyy	N/A	N/A	Lateral
7	M13	Main Stringer Start	0.25	Lbyy	N/A	N/A	Lateral
8	M10	Main Stringer	0.25	Lbyy	N/A	N/A	Lateral

Basic Load Cases

	BLC Description	Category	Y Gravity	Point	Distributed
1	DL	DL	-1		
2	Super Imposed	DL			3
3	LL Full	LL			3
4	0.5 LL	LL			6
5	Mrailing	LL		4	
6		None		1	1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL+SI+LL Full	Yes	Y	1	1	2	1	3	1	5	1
2	DL+SI+0.5 LL	Yes	Y	1	1	2	1	4	1	5	1
3	DL ONLY	Yes	Y	DL	1						

Eastern Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

SOFTWARE: RISA 3D
RISA-3D V.22.0.1
RESULTS

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N11	max	-0.372	3	0	3	0.306	1	0	3	0	3	0	3
2		min	-1.021	1	0	1	0.123	3	0	1	0	1	0	1
3	N13	max	0.212	1	3.112	1	0.273	1	0	3	1.227	1	0	3
4		min	0.112	3	1.508	3	0.114	3	0	1	0.495	3	0	1
5	N14	max	0.25	1	5.022	1	-1.281	3	5.874	1	-0.598	3	0.97	2
6		min	0.133	2	2.013	3	-3.249	1	2.389	3	-1.09	1	-0.26	3
7	N16	max	0.159	1	4.006	1	2.669	1	0	3	0	3	2.868	2
8		min	0.095	3	1.592	3	1.044	3	0	1	0	1	0.998	3
9	Totals:	max	0	3	12.14	1	0	1						
10		min	-0.4	2	5.113	3	0	3						

Envelope Node Displacements

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
1	N1	max	0	2	0	3	0	3	-2.747e-4	3	9.97e-5	1	8.736e-6	3
2		min	0	3	0	1	0	1	-6.767e-4	1	5.467e-5	3	-3.602e-5	2
3	N2	max	0.161	1	-0.061	3	-0.038	3	3.577e-4	1	-1.73e-3	3	-2.051e-3	3
4		min	0.036	3	-0.152	1	-0.095	1	2.09e-5	2	-4.184e-3	1	-5.375e-3	1
5	N3	max	0.295	1	-0.001	3	-0.097	3	2.301e-4	1	-2.307e-3	3	-2.098e-3	3
6		min	0.091	3	-0.002	1	-0.24	1	-2.619e-5	2	-5.722e-3	1	-5.212e-3	1
7	N6	max	0.296	1	-0.191	3	0.315	1	2.065e-4	1	-3.867e-3	3	-3.285e-3	3
8		min	0.091	3	-0.48	1	0.126	3	-3.673e-4	2	-9.608e-3	1	-8.382e-3	1
9	N9	max	0.095	1	-0.194	3	0.316	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10		min	0.01	3	-0.484	1	0.126	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	N13	max	0	3	0	3	0	[illegible]	[illegible]	[illegible]	[illegible]	3	-4.504e-4	3
12		min	0	1	0	1	0	[illegible]	[illegible]	[illegible]	[illegible]	1	-2.271e-3	1
13	N14	max	0	2	0	3	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
14		min	0	1	0	1	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
15	N12	max	0.011	2	-0.01	3	0	3	-3.142e-3	3	-6.527e-4	3	-9.123e-5	3
16		min	0.002	3	-0.024	1	0	[illegible]	[illegible]	[illegible]	[illegible]	2	-2.623e-4	2
17	N10	max	0.295	1	-0.058	3	-0.038	[illegible]	[illegible]	[illegible]	[illegible]	3	-2.767e-3	3
18		min	0.091	3	-0.145	1	-0.094	[illegible]	[illegible]	[illegible]	[illegible]	1	-6.945e-3	1
19	N11	max	0	1	-0.001	3	0	[illegible]	[illegible]	[illegible]	[illegible]	3	7.835e-3	1
20		min	0	3	-0.002	1	0	[illegible]	[illegible]	[illegible]	[illegible]	1	2.652e-3	3
21	N15	max	0.296	1	-0.26	3	0.517	[illegible]	[illegible]	[illegible]	[illegible]	3	-3.305e-3	3
22		min	0.091	3	-0.657	1	0.207	[illegible]	[illegible]	[illegible]	[illegible]	1	-8.433e-3	1
23	N16	max	0	3	0	3	0	3	-3.149e-3	3	-6.514e-4	3	0	3
24		min	0	1	0	1	0	1	-7.821e-3	1	-3.836e-3	2	0	2

TABLE R301.7ALLOWABLE DEFLECTION OF STRUCTURAL MEMBERS[b, c]

All other structural members	$L/240$

b For cantilever members, L shall be taken as twice the length of the cantilever.

Δmax = 0.65"

ΔAllowable = 7ft x 2 / 240 *12
ΔAllowable = 0.70" > 0.65" (OK)

Envelope Maximum Member Section Forces

	Member		Axial[k]	Loc[ft]	LC	y Shear[k]	Loc[ft]	LC	z Shear[k]	Loc[ft]	LC	Torque[k-ft]	Loc[ft]	LC	y-y Moment[k-ft]	Loc[ft]	LC	z-z Moment[k-ft]	Loc[ft]	LC
1	M3	max	0	7.16	3	2.782	0	1	0.587	1.865	1	0.363	1.939	2	6.184	1.939	1	8.634	0	1
2		min	-0.809	0	1	0	7.16	1	-2.664	1.939	1	-0.028	5.37	2	-2.955	5.37	1	-0.073	5.37	3
3	M6	max	5.486	0	1	2.38	0	1	0.443	13.22	1	0.502	13.22	2	3.746	13.22	1	6.686	0	1
4		min	1.145	13.22	3	-1.483	13.22	1	0.129	0	2	-1.415	0	2	-0.789	0	1	-3.008	8.125	1
5	M7	max	3.249	2	1	0.373	0	1	0.421	2	1	-0.979	2	3	3.913	2	1	0.914	0	2
6		min	1.281	0	3	0.103	2	3	0.159	0	3	-2.146	0	1	1.199	0	3	0.019	2	3
7	M8	max	-1.044	2	3	0.335	0	1	0.395	2	1	0.335	2	2	-0.984	2	3	0.232	0	2
8		min	-2.669	0	1	0.14	2	3	0.101	0	3	-0.354	0	1	-3.062	0	1	-0.66	2	1
9	M9	max	-0.776	0	3	1.79	0	1	0.362	5.993	1	-0.549	0	2	1.609	12.241	2	-0.102	0	2
10		min	-4.478	12.241	1	-1.736	12.241	1	0.096	0	3	-2.375	12.241	2	-1.662	0	1	-6.223	6.248	1
11	M12	max	3.112	14.583	1	1.021	6.988	1	0.285	14.583	1	1.227	14.583	1	0	14.583	3	1.46	7.14	1
12		min	0	0	1	0.11	7.14	3	-0.307	0	1	0	0	1	-2.143	6.988	1	-7.134	6.988	1
13	M13	max	5.022	0.25	1	0.25	0.25	1	-1.281	0.25	3	-0.598	0.25	3	6.686	0	1	1.003	0	2
14		min	2.008	0	3	0.133	0	2	-3.248	0	1	-1.09	0	1	2.389	0.25	3	-0.26	0.25	3
15	M10	max	-1.044	0.25	3	-1.582	0	3	0.173	0.25	1	-0.998	0.25	3	0	0.25	3	0	0.25	3
16		min	-2.669	0	1	-3.972	0.25	1	0.097	0	3	-2.868	0	2	-0.043	0	1	-0.992	0	1

Envelope Member End Reactions

	Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
1	M3	I	max	-0.259	3	2.782	1	0.587	1	0.291	2	1.227	1	8.634	1
2			min	-0.809	1	1.185	3	0.238	3	-0.023	3	0.495	3	3.436	3
3		J	max	0	3	0	3	0	3	0	3	0	3	0	3
4			min	0	1	0	1	0	1	0	1	0	1	0	1
5	M6	I	max	5.486	1	2.38	1	0.243	1	-0.15	3	-0.313	2	6.686	1
6			min	2.18	3	0.961	3	0.129	2	-1.415	2	-0.789	1	2.709	3
7		J	max	2.911	1	-0.591	3	0.443	1	0.502	2	3.746	1	0.914	2
8			min	1.145	3	-1.483	1	0.163	3	-0.15	3	1.541	3	0.263	3
9	M7	I	max	3.249	1	0.373	1	0.421	1	-0.979	3	3.071	1	0.914	2
10			min	1.281	3	0.141	3	0.159	3	-2.146	1	1.199	3	0.263	3
11		J	max	3.249	1	0.335	1	0.421	1	-0.979	3	3.913	1	0.293	2
12			min	1.281	3	0.103	3	0.159	3	-2.146	1	1.518	3	0.019	3
13	M8	I	max	-1.044	3	0.335	1	0.395	1	0.335	2	-1.186	3	0.232	2
14			min	-2.669	1	0.179	3	0.101	3	-0.354	1	-3.062	1	-0.028	1
15		J	max	-1.044	3	0.297	1	0.395	1	0.335	2	-0.984	3	-0.102	2
16			min	-2.669	1	0.14	3	0.101	3	-0.354	1	-2.273	1	-0.66	1
17	M9	I	max	-0.776	3	1.79	1	0.362	1	-0.549	2	-0.619	3	-0.102	2
18			min	-2.02	1	0.716	3	0.096	3	-1.59	1	-1.662	1	-0.66	1

Envelope Member End Reactions (Continued)

	Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
19		J	max	-1.765	3	-0.702	3	0.162	1	-0.832	3	1.609	2	-0.396	3
20			min	-4.478	1	-1.736	1	0.096	3	-2.375	2	0.551	3	-0.992	1
21	M12	I	max	0	3	1.021	1	-0.123	3	0	3	0	3	0	3
22			min	0	1	0.372	3	-0.307	1	0	1	0	1	0	1
23		J	max	3.112	1	0.196	1	0.285	1	1.227	1	0	3	0	3
24			min	1.508	3	0.11	3	0.116	3	0.495	3	0	1	0	1
25	M13	I	max	5.017	1	0.25	1	-1.281	3	-0.598	3	6.686	1	1.003	2
26			min	2.008	3	0.133	2	-3.248	1	-1.09	1	2.709	3	-0.219	3
27		J	max	5.022	1	0.25	1	-1.281	3	-0.598	3	5.874	1	0.97	2
28			min	2.013	3	0.133	2	-3.248	1	-1.09	1	2.389	3	-0.26	3
29	M10	I	max	-1.044	3	-1.582	3	0.173	1	-0.998	3	-0.024	3	-0.396	3
30			min	-2.669	1	-3.967	1	0.097	3	-2.868	2	-0.043	1	-0.992	1
31		J	max	-1.044	3	-1.587	3	0.173	1	-0.998	3	0	3	0	3
32			min	-2.669	1	-3.972	1	0.097	3	-2.868	2	0	1	0	1

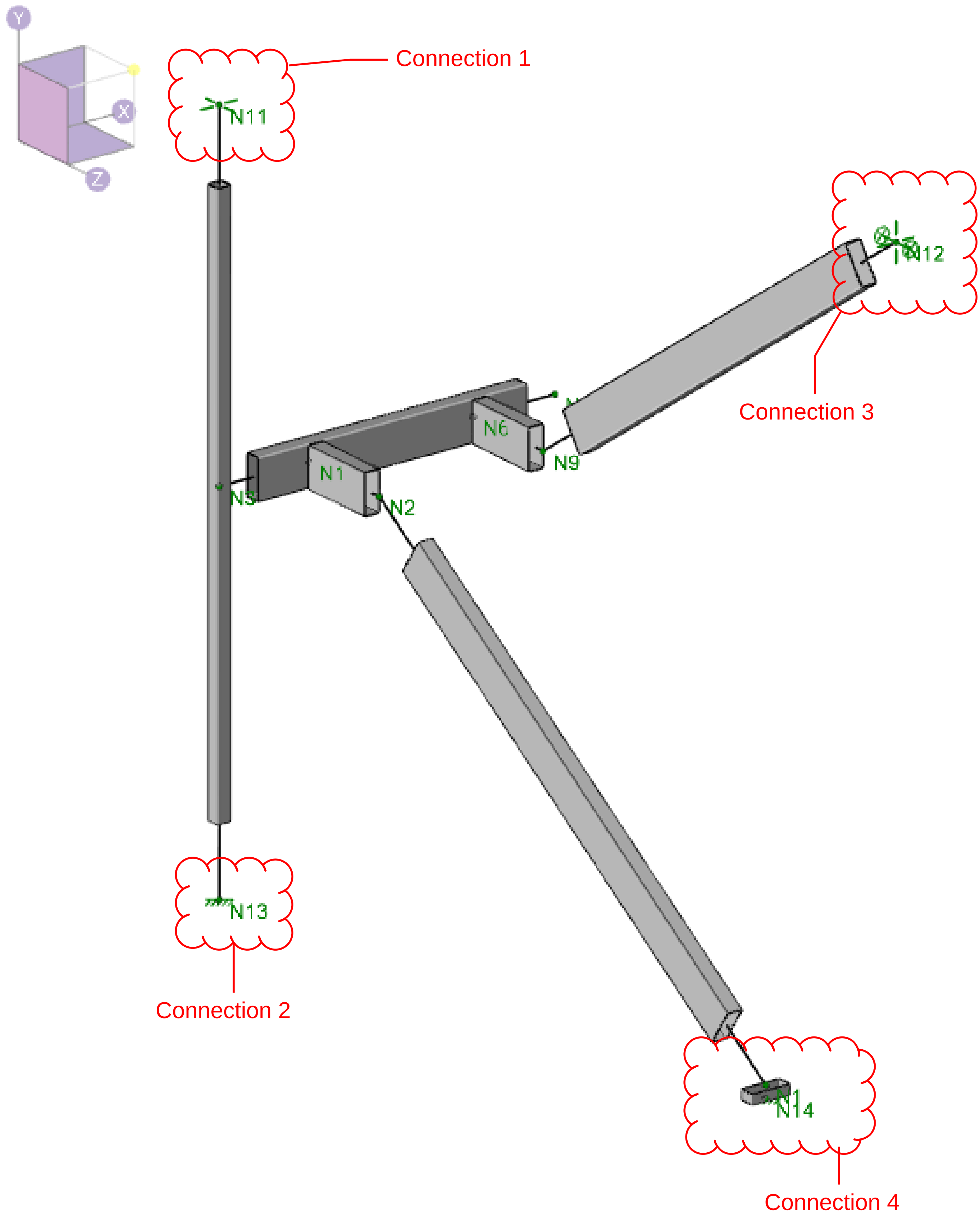
Envelope AISC 15TH (360-16): ASD Member Steel Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	M3	HSS8X4X4	0.549	1.939	1	0.106	5.37	z	1	120.474	144.335	17.777	30.529	2.333	H1-1b
2	M6	HSS8X4X4	0.299	0	1	0.104	0	y	2	77.952	144.335	17.777	30.529	2.304	H1-1b
3	M7	HSS8X4X4	0.233	2	1	0.131	2	z	1	142.315	144.335	17.777	30.529	1.624	H1-1b
4	M8	HSS8X4X4	0.182	0	1	0.034	2	z	1	142.315	144.335	17.777	30.529	1.595	H1-1b
5	M9	HSS8X4X4	0.252	7.268	1	0.153	12.241	y	1	85.115	144.335	17.777	30.529	1.116	H1-1b
6	M12	HSS4X4X8	0.526	6.988	1	0.084	14.583	z	1	58.405	165.82	17.675	17.675	2.419	H1-1b
7	M13	HSS8X4X4	0.4	0	1	0.186	0.25	z	1	144.304	144.335	17.777	30.529	1.133	H1-1b
8	M10	HSS8X4X4	0.044	0	1	0.218	0.25	y	1	144.304	144.335	17.777	30.529	1.666	H1-1b

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

CONNECTIONS DESIGN

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com



Eastern
Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

Connection 1

loads:
N11
Rx = 1.021 kips = 1021 #
Ry = 0
Rz = 0.306 kips = 306 #

load processing:

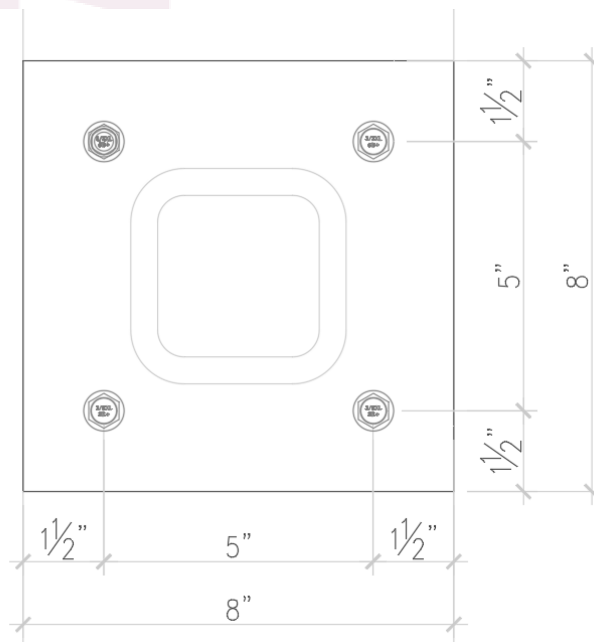
Resultant Shear = $\sqrt{(Rx)^2 + (Rz)^2}$
Resultant Shear = 1066#

Shear per bolt = 1066#÷4 bolts
Shear per bolt = 266.5#

Connection to concrete beam 2B-8

Use (4) Ø3/8" screw bolt+ by dewalt w/2-1/2" min. embedment into concrete, 5" min spacing & 1-1/2" min. edge distance.

Sallow = 1365# x 0.39 x 0.71 = 377.96# > Sbolt (OK)



Allowable Load Capacities for Screw-Bolt+ in Normal-Weight Concrete[1,2,3,4,5]

Nominal Anchor Diameter in.	Minimum Nominal Embedment Depth in. (mm)	Minimum Concrete Compressive Strength									
		f'c = 2,500 psi (17.3 MPa)		f'c = 3,000 psi (20.7 MPa)		f'c = 4,000 psi (27.6 MPa)		f'c = 6,000 psi (41.4 MPa)		f'c = 8,000 psi (55.2 MPa)	
		Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)
1/4	1 (25)	330 (1.5)	415 (1.8)	350 (1.6)	440 (2.0)	385 (1.7)	480 (2.1)	430 (1.9)	520 (2.3)	430 (1.9)	520 (2.3)
	1-5/8 (41)	710 (3.2)	415 (1.8)	750 (3.3)	440 (2.0)	815 (3.6)	480 (2.1)	815 (3.6)	520 (2.3)	815 (3.6)	520 (2.3)
	2-1/2 (64)	915 (4.1)	505 (2.2)	965 (4.3)	535 (2.4)	1,050 (4.7)	585 (2.6)	1,070 (4.8)	635 (2.8)	1,070 (4.8)	635 (2.8)
3/8	1-1/2 (38)	660 (2.9)	890 (4.0)	720 (3.2)	975 (4.3)	835 (3.7)	1,125 (5.0)	1,020 (4.5)	1,375 (6.1)	1,020 (4.5)	1,590 (7.1)
	2 (51)	920 (4.1)	1,080 (4.8)	1,005 (4.5)	1,185 (5.3)	1,160 (5.2)	1,365 (6.1)	1,180 (5.2)	1,585 (7.1)	1,365 (6.1)	1,585 (7.1)
	2-1/2 (64)	1,295 (5.8)	1,080 (4.8)	1,415 (6.3)	1,185 (5.3)	1,600 (7.1)	1,365 (6.1)	1,615 (7.2)	1,585 (7.1)	1,855 (8.3)	1,585 (7.1)
	3-1/4 (83)	1,855 (8.3)	1,580 (7.0)	2,035 (9.1)	1,735 (7.7)	2,265 (10.1)	2,000 (8.9)	2,265 (10.1)	2,140 (9.5)	2,590 (11.5)	2,140 (9.5)
	4-1/2 (114)	2,725 (12.1)	1,580 (7.0)	2,985 (13.3)	1,735 (7.7)	3,450 (15.3)	2,000 (8.9)	3,770 (16.8)	2,140 (9.5)	3,770 (16.8)	2,140 (9.5)
1/2	1-3/4 (44)	710 (3.2)	1,495 (6.7)	780 (3.5)	1,640 (7.3)	900 (4.0)	1,895 (8.4)	1,100 (4.9)	2,320 (10.3)	1,100 (4.9)	2,675 (11.9)
	2-1/2 (64)	1,670 (7.4)	2,010 (8.9)	1,830 (8.1)	2,200 (9.8)	2,115 (9.4)	2,540 (11.3)	2,115 (9.4)	2,885 (12.8)	2,115 (9.4)	2,885 (12.8)
	3 (76)	2,140 (9.5)	2,010 (8.9)	2,345 (10.4)	2,200 (9.8)	2,690 (11.9)	2,540 (11.3)	2,690 (11.9)	2,885 (12.8)	2,690 (11.9)	2,885 (12.8)
	4-1/4 (108)	3,315 (14.7)	2,350 (10.5)	3,630 (16.1)	2,575 (11.5)	4,120 (18.3)	2,970 (13.2)	4,120 (18.3)	3,380 (15.0)	4,120 (18.3)	3,380 (15.0)
	5-1/2 (140)	3,935 (17.5)	2,350 (10.5)	4,310 (19.2)	2,575 (11.5)	4,975 (22.1)	2,970 (13.2)	5,330 (23.7)	3,380 (15.0)	5,330 (23.7)	3,380 (15.0)
5/8	2-1/2 (64)	1,435 (6.4)	2,655 (11.8)	1,570 (7.0)	2,910 (12.9)	1,815 (8.1)	3,355 (14.9)	2,220 (9.9)	4,110 (18.3)	2,220 (9.9)	4,295 (19.1)
	3-1/4 (83)	2,440 (10.9)	3,015 (13.4)	2,670 (11.9)	3,305 (14.7)	3,085 (13.7)	3,815 (17.0)	3,085 (13.7)	4,295 (19.1)	3,085 (13.7)	4,295 (19.1)
	4 (102)	2,940 (13.1)	3,015 (13.4)	3,225 (14.3)	3,305 (14.7)	3,720 (16.5)	3,815 (16.9)	3,830 (17.0)	4,295 (19.1)	4,150 (18.5)	4,295 (19.1)
	5 (127)	3,615 (16.1)	3,420 (15.2)	3,960 (17.6)	3,745 (16.7)	4,570 (20.3)	4,325 (19.2)	4,825 (21.5)	4,870 (21.7)	5,570 (24.8)	4,870 (21.7)
	6-1/4 (159)	5,130 (22.8)	3,420 (15.2)	5,620 (25.0)	3,745 (16.7)	6,490 (28.9)	4,325 (19.2)	7,945 (35.3)	4,870 (21.7)	7,945 (35.3)	4,870 (21.7)
3/4	2-1/2 (64)	1,510 (6.7)	2,905 (12.9)	1,655 (7.4)	3,180 (14.1)	1,910 (8.5)	3,675 (16.3)	2,340 (10.4)	4,500 (20.0)	2,340 (10.4)	5,195 (23.1)
	4-1/4 (108)	2,975 (13.2)	4,265 (19.0)	3,260 (14.5)	4,670 (20.8)	3,765 (16.7)	5,395 (24.0)	4,435 (19.7)	6,070 (27.0)	5,125 (22.8)	6,070 (27.0)
	5 (127)	4,755 (21.2)	4,265 (19.0)	5,210 (23.2)	4,670 (20.8)	6,015 (26.8)	5,395 (24.0)	7,365 (32.8)	6,070 (27.0)	7,365 (32.8)	6,070 (27.0)
	6-1/4 (159)	5,125 (22.8)	4,265 (19.0)	5,615 (25.0)	4,670 (20.8)	6,480 (28.8)	5,395 (24.0)	7,940 (35.3)	6,070 (27.0)	7,940 (35.3)	6,070 (27.0)

1. Tabulated load values are for anchors installed in uncracked concrete. Concrete compressive strength must be at the specified minimum at the time of installation.
2. Allowable load capacities are calculated using an applied safety factor of 4.0 to average ultimate load capacities.
3. Allowable load capacities must be multiplied by reduction factors when anchor spacing or edge distances are less than critical distances.
4. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
5. For lightweight concrete multiply tabulated allowable load values by a reduction factor of 0.60.

Edge Distance Reduction Factors - Shear (F_{VC})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Min. Edge Distance c_{min} (in)		1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4
Edge Distance (inches)	1-1/2	0.58	0.63	0.59	0.40	0.37	0.39	0.31	0.32	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	0.68	0.73	0.69	0.46	0.43	0.45	0.36	0.38	0.35	0.31	0.36	0.30	0.31	0.27	0.26	0.32	0.25	0.26	0.26	0.22	0.22	0.23
	2	0.78	0.84	0.78	0.53	0.49	0.52	0.41	0.43	0.41	0.35	0.41	0.35	0.36	0.30	0.29	0.37	0.29	0.30	0.30	0.25	0.26	0.27
	2-1/4	0.87	0.94	0.88	0.59	0.55	0.58	0.46	0.48	0.46	0.40	0.46	0.39	0.40	0.34	0.33	0.41	0.32	0.33	0.33	0.28	0.29	0.30
	2-1/2	0.97	1.00	0.98	0.66	0.61	0.64	0.51	0.54	0.51	0.44	0.51	0.43	0.45	0.38	0.36	0.46	0.36	0.37	0.37	0.31	0.32	0.33
	2-3/4	1.00	1.00	1.00	0.73	0.67	0.71	0.56	0.59	0.56	0.49	0.56	0.48	0.49	0.42	0.40	0.51	0.40	0.41	0.41	0.34	0.35	0.37
	3	1.00	1.00	1.00	0.79	0.73	0.77	0.61	0.64	0.61	0.53	0.61	0.52	0.54	0.46	0.44	0.55	0.43	0.45	0.44	0.38	0.39	0.40
	3-1/2	1.00	1.00	1.00	0.92	0.85	0.90	0.72	0.75	0.71	0.62	0.72	0.61	0.63	0.53	0.51	0.64	0.50	0.52	0.52	0.44	0.45	0.47
	4	1.00	1.00	1.00	1.00	0.97	1.00	0.82	0.86	0.81	0.71	0.82	0.69	0.72	0.61	0.58	0.74	0.57	0.59	0.59	0.50	0.51	0.53
	4-1/2	1.00	1.00	1.00	1.00	1.00	1.00	0.92	0.97	0.91	0.80	0.92	0.78	0.81	0.68	0.66	0.83	0.65	0.67	0.67	0.56	0.58	0.60
	5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.89	1.00	0.87	0.90	0.76	0.73	0.92	0.72	0.74	0.74	0.63	0.64	0.66
	5-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	0.95	0.99	0.84	0.80	1.00	0.79	0.82	0.82	0.69	0.71	0.73
	6	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.88	1.00	0.86	0.89	0.89	0.75	0.77	0.80
	6-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.95	1.00	0.93	0.97	0.96	0.81	0.84	0.86
	7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.88	0.90	0.93
	7-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.96	1.00
	8	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Spacing Reduction Factors - Shear (F_{VS})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Minimum Spacing s_{min} (in)		1-1/2	1-1/2	1-1/2	2	2	2	2	2	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	3	3	3	3
Spacing Distance (inches)	1-1/2	0.60	0.60	0.60	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	0.61	0.62	0.61	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2	0.63	0.64	0.63	0.59	0.58	0.59	0.57	0.57	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/4	0.65	0.66	0.65	0.60	0.59	0.60	0.58	0.58	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/2	0.66	0.67	0.66	0.61	0.60	0.61	0.59	0.59	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-3/4	0.68	0.69	0.68	0.62	0.61	0.62	0.59	0.60	0.59	0.58	0.59	0.58	0.58	0.57	0.57	0.58	0.57	0.57	-	-	-	-
	3	0.69	0.71	0.70	0.63	0.62	0.63	0.60	0.61	0.60	0.59	0.60	0.59	0.59	0.58	0.57	0.59	0.57	0.57	0.57	0.56	0.56	0.57
	3-1/2	0.73	0.74	0.73	0.65	0.64	0.65	0.62	0.63	0.62	0.60	0.62	0.60	0.60	0.59	0.59	0.61	0.58	0.59	0.59	0.57	0.57	0.58
	4	0.76	0.78	0.76	0.68	0.66	0.67	0.64	0.64	0.64	0.62	0.64	0.62	0.62	0.60	0.60	0.62	0.60	0.60	0.60	0.58	0.59	0.59
	4-1/2	0.79	0.81	0.79	0.70	0.68	0.69	0.65	0.66	0.65	0.63	0.65	0.63	0.63	0.61	0.61	0.64	0.61	0.61	0.61	0.59	0.60	0.60
	5	0.82	0.85	0.83	0.72	0.70	0.71	0.67	0.68	0.67	0.65	0.67	0.64	0.65	0.63	0.62	0.65	0.62	0.62	0.62	0.60	0.61	0.61
	5-1/2	0.86	0.88	0.86	0.74	0.72	0.74	0.69	0.70	0.69	0.66	0.69	0.66	0.66	0.64	0.63	0.67	0.63	0.64	0.64	0.61	0.62	0.62
	6	0.89	0.92	0.89	0.76	0.74	0.76	0.70	0.71	0.70	0.68	0.70	0.67	0.68	0.65	0.65	0.68	0.64	0.65	0.65	0.63	0.63	0.63
	6-1/2	0.92	0.95	0.92	0.79	0.76	0.78	0.72	0.73	0.72	0.69	0.72	0.69	0.69	0.66	0.66	0.70	0.66	0.66	0.66	0.64	0.64	0.64
	7	0.95	0.99	0.96	0.81	0.78	0.80	0.74	0.75	0.74	0.71	0.74	0.70	0.71	0.68	0.67	0.71	0.67	0.67	0.67	0.65	0.65	0.66
	7-1/2	0.99	1.00	0.99	0.83	0.80	0.82	0.76	0.77	0.75	0.72	0.76	0.72	0.72	0.69	0.68	0.73	0.68	0.69	0.69	0.66	0.66	0.67
	8	1.00	1.00	1.00	0.85	0.82	0.84	0.77	0.79	0.77	0.74	0.77	0.73	0.74	0.70	0.69	0.75	0.69	0.70	0.70	0.67	0.67	0.68
	9	1.00	1.00	1.00	0.90	0.87	0.89	0.81	0.82	0.80	0.77	0.81	0.76	0.77	0.73	0.72	0.78	0.72	0.72	0.72	0.69	0.69	0.70
	10	1.00	1.00	1.00	0.94	0.91	0.93	0.84	0.86	0.84	0.80	0.84	0.79	0.80	0.75	0.74	0.81	0.74	0.75	0.75	0.71	0.71	0.72
	11	1.00	1.00	1.00	0.98	0.95	0.97	0.87	0.89	0.87	0.82	0.87	0.82	0.83	0.78	0.77	0.84	0.76	0.77	0.77	0.73	0.74	0.74
	12	1.00	1.00	1.00	1.00	0.99	1.00	0.91	0.93	0.91	0.85	0.91	0.85	0.86	0.80	0.79	0.87	0.79	0.80	0.80	0.75	0.76	0.77
	13	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.96	0.94	0.88	0.94	0.88	0.89	0.83	0.82	0.90	0.81	0.82	0.82	0.77	0.78	0.79
	14	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	0.97	0.91	0.98	0.90	0.92	0.85	0.84	0.93	0.84	0.85	0.85	0.79	0.80	0.81
	15	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	1.00	0.93	0.95	0.88	0.86	0.96	0.86	0.87	0.87	0.81	0.82	0.83
	16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	0.96	0.98	0.91	0.89	0.99	0.88	0.90	0.90	0.83	0.84	0.85
	17	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	0.93	0.91	1.00	0.91	0.92	0.92	0.86	0.86	0.88
	18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.94	1.00	0.93	0.95	0.94	0.88	0.89	0.90
	19	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.96	1.00	0.95	0.97	0.97	0.90	0.91	0.92
	20	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	0.98	1.00	0.99	0.92	0.93	0.94
	21	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.95	0.97
	22	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.97	0.99
	23	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.99	1.00
	24	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

Connection 2

loads:
N13
Rx = 0.212 kips = 212 # x 1.6 = 339.2# (LRFD)
Ry = 3.112 kips = 3112 x 1.6 = 4979.2# (LRFD)
Rz = 0.273 kips = 273# x 1.6 = 436.8# (LRFD)
My = 1.227 k-ft= 14724 #.in x 1.6 = 23558.4 #.in (LRFD)

load processing:
(See next sheet printout)

Connection to slab

Use (4) Ø3/8" screw bolt+ by dewalt w/3-1/4" min. embedment into concrete, 6" min spacing & 6" min. edge distance.

Column Slab Conncection

3/16/2026

1.Project Information

Company:	
Project Engineer:	Maria Peroza
Address:	Florida 33122
Phone:	M: P:
Email:	maria.e@easterneg.com
Project Name:	Untitled
Project Address:	Untitled

Notes:

Connection 2 - Connection to slab

2.Selected Anchor Information

Selected Anchor :	Screw-Bolt+					
Brand:	DEWALT					
Material:	3/8" Ø Low Carbon Steel					
Embedment:	h_{ef}	2.39	in	h_{nom}	3.25	in
Approval:	ICC-ES ESR-3889					
Issued/Revision:	Nov,2024	-				
Drill method:	Hammer Drilled					

3.Design Principles

Design Method:	ACI 318-19	
Load Combinations:	Section 5.3	User Defined Loads

4.Base Material Information

Concrete:

Type:	Cracked		Normal Weight Concrete					
Strength	5000	psi						

Reinforcement:

Edge Reinforcement	None or < #4 Rebar			
Spacing	Tension	No (Condition B)	Shear	No (Condition B)
Controls Breakout	Tension	False	Shear	False

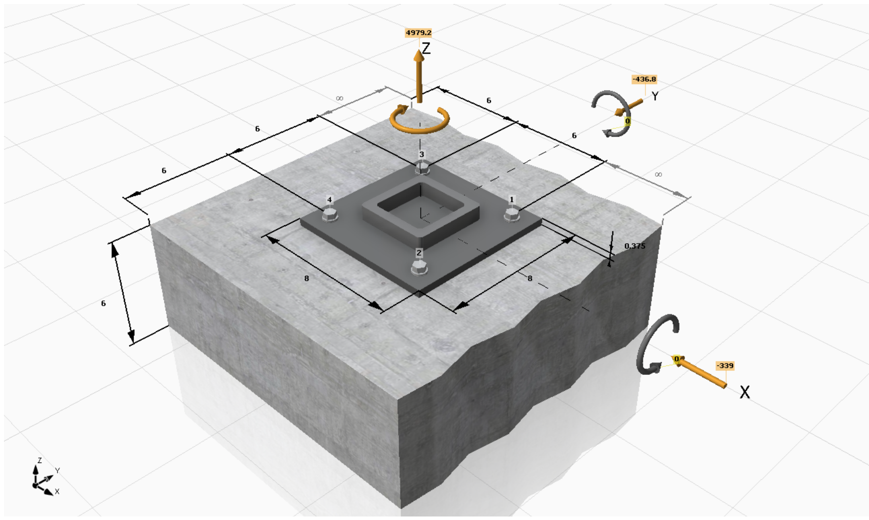
Base Plate:

Sizing	Thickness	0.375	in	Length	8	in	Width 8	in
Standoff	None			Height	0	in		
Strength	36000	psi						

Profile: HSS Rectangular 4 X 4 X 0.500

Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

5.Geometric Conditions



h_{slab}	6	in	h_{min}	5.000	in
Edge Cx-	6	in	c_{min}	1.500	in
Edge Cx+	∞	in	c_{ac}	7.800	in
Edge Cy-	6	in	s_{min}	2.000	in
Edge Cy+	∞	in			

6.Summary Results

Tension Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	1244.80	5674.00	0.219	OK	
Concrete Breakout Strength:	4979.00	9740.00	0.511	OK	Controls

Shear Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	1525.00	2607.00	0.585	OK	
Concrete Breakout Strength:	2396.00	3872.00	0.619	OK	Controls
Pryout Strength	1525.00	3109.00	0.491	OK	

7.Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

- The results of the calculations carried out by means of the DDA 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 design professional/engineer, particularly with regard to compliance with applicable standards, norms and permits, prior to using them for your specific project. The DDA Software serves only as an aid to interpret standards, 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.
- Verification of a sufficiently stiff base plate in respect to base plate deformations is not performed by the DDA Software, this check should be done by a design professional/engineer outside of the DDA Software. The DDA base plate thickness analysis is based on a proof of stresses, the base plate is assumed to not deform under load actions.

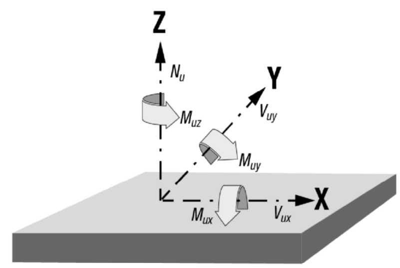
8.Load Condition

Design Loads / Actions

Nu	4979.2	lb	Vux	-339.0	lb	Vuy	-436.8	lb
Muz	-23559.0	in-lb	Mux	0.0	in-lb	Muy	0.0	in-lb

Consider Load Reversal

X Direction	0%	Y Direction	0%



9.Load Distribution

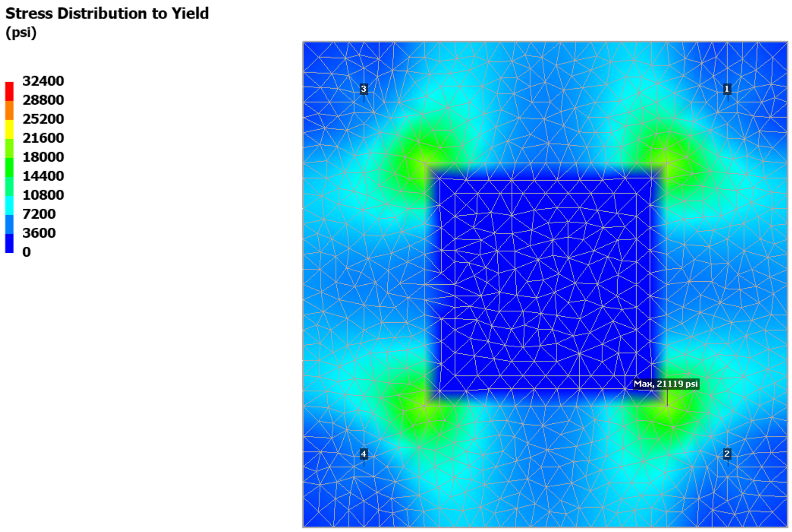
Max. concrete compressive strain:	0.000	%	Anchor Eccentricity					
Max. concrete compressive stress:	0.000	psi	ex	0	in	ey	0	in
Resulting tension force:	4979.200	lb	Profile Eccentricity					
Resulting compression force:	0.000	lb	ex	0	in	ey	0	in

Resulting anchor forces / Load distribution

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb) Shear X	Shear Y	Anchor Coordinates (in) X	Y
1	1244.80	1412.2	896.9	-1090.8	3.000	3.000
2	1244.80	1525.5	-1066.4	-1090.8	3.000	-3.000
3	1244.80	1251.2	896.9	872.4	-3.000	3.000
4	1244.80	1377.8	-1066.4	872.4	-3.000	-3.000

10.Base Plate Analysis

Stress Distribution



Details

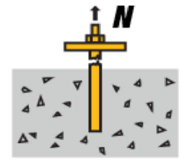
Material		A36	
Yield Stress	f_y	36000	psi
Elastic Modulus	E	29000000	psi
Poisson's Ratio	ν	0.27	
Safety Factor	γ	0.9	
Profile Type		HSS Rectangular	4 X 4 X 0.500
Max Stress	Σ	21119	psi
Base Plate Thickness	t_p	0.375	in

Base Plate Utilization

$\Sigma / (f_y * \gamma)$ 65.2%

11.Design Proof Tension Loading

Steel Strength



ACI 318-19 17.6.1

Variables

N_{sa} (lb)	φ
8730.000	0.65

Results

φN_{sa}	=	5674.0	lb	Table 17.5.2
N_{ua}	=	1244.8	lb	
Utilization	=	21.9%		

Concrete Breakout Strength

ACI 318-19 17.6.2

Equations

$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$ Eqn. 17.6.2.1b

$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
173.449	51.409	1.000	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
7.800	17.000	1.000	5000	2.39	6

N_b (lb)	φ
4441.510	0.65

Results

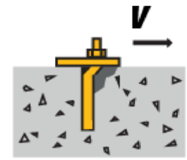
φN_{cbg}	=	9740	lb	Table 17.5.2
N_{ua}	=	4979.0	lb	
Utilization	=	51.1%		

12.Design Proof Shear Loading

Reference

Steel Strength

ACI 318-19 17.7.1



Variables

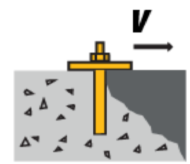
V_{sa} (lb)	φ
4345.00	0.60

Results

φv_{sa}	=	2607	lb	
V_{ua}	=	1525	lb	Table 17.5.2
Utilization	=	58.5%		

Concrete Breakout Strength

ACI 318-19 17.7.2



Equations

$$V_{cbg} = (A_{Vc} / A_{Vc0}) .\ \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,v} \cdot V_b$$

Eqn. 17.7.2.1b

$$V_b = (7 . (l_e / d_a)^{0.2} . (d_a)^{0.5}) . \lambda_a . (f'_c)^{0.5} . (c_{a1})^{1.5}$$

Eqn. 17.7.2.2.1a

Variables

A_{Vc} (in²)	A_{Vc0} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
126.000	162.000	1.000	0.900	1.000	1.225

$\Psi_{\alpha,V}$	l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)
1.000	2.390	0.375	1.000	5000.000	6.000

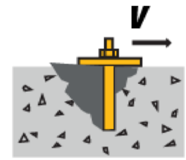
V_b (lb)	φ	V_{cbg} (lb)
6452.067	0.70	5531.495

Results

φV_{cbg}	=	3872	lb	
Direction	=	Y-		
V_{ua}	=	2396	lb	Table 17.5.2
Utilization	=	61.9%		

Shear Breakout Case 1 Controls

Pryout Strength
ACI 318-19 17.7.3



Equations

$V_{cpg} = k_{cp} \,.\, N_{cpg}$ Eqn. 17.7.3.1b

$N_{cbg} = (A_{Nc} / A_{Nc0}) \,.\, \Psi_{ec,N} \,.\, \Psi_{ed,N} \,.\, \Psi_{c,N} \,.\, \Psi_{cp,N} \,.\, N_b$ Eqn. 17.6.2.1b

$N_b = k_c \,.\, \lambda_a \,.\, (f'_c)^{0.5} \,.\, h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
51.409	51.409	1.000	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
7.800	17.000	1.000	2.39	5000.000	6.000

N_b (lb)	k_{cp}	N_{cpg} (lb)	φ
4441.511	1.000	4441.511	0.70

Results

φV_{cpg}	=	3109.00	lb	Table 17.5.2
V_{ua}	=	1525	lb	
Utilization	=	49.1%		

13.Interaction of Tension and Shear Loads

Reference

ACI 318-19 17.8

Equations

$(N_{ua} / \varphi \cdot N_n)^{5/3} + (V_{ua} / \varphi \cdot V_n)^{5/3} <= 1.0$ Eqn. 17.8.3

Variables

$N_{ua} / \varphi \cdot N_n$	$V_{ua} / \varphi \cdot V_n$
0.511	0.619

Results

0.776 ≤ 1.0

Status : OK

ANCHOR DESIGN CRITERIA IS SATISFIED

Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 4.00$ Width of Tube Steel (in)

$d := 4.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 1.227$ Moment (Kips-ft)

$V_{weld} := 0.273$ Shear (Kips)

$P_{weld} := 3.112$ Tension or Compression (Kips)

$T_{weld} := 0.0$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while} \sqrt{\left[\frac{P_{weld}}{2(b+d)\cdot x} + \frac{M_{weld}\cdot 12}{\left(b\cdot d + \frac{d^2}{3}\right)\cdot x}\right]^2 + \left[\frac{1.5V_{weld}}{2(b+d)\cdot x} + \frac{T_{weld}\cdot 12\cdot\sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6}\cdot x}\right]^2} \geq F_v \\ \quad x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.05$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 1.07$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $A_{weld} = 0.80$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $J_{weld} = 4.27$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$

$f_b = 13.80$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$

$f_v = 0.51$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$

$f_a = 3.89$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^{\,2}}$$

$f_{weld} = 17.70$ ksi

$$STRESS := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{array} \right.$$

$STRESS = \text{"OK"}$

$$W_{weld} := \left| \begin{array}{l} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{array} \right.$$

$W_{weld} = \text{"3/16"}$

Eastern
Engineering Group

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

Connection 3

loads:
N16
Rx = 0.159 kips = 159 # x 1.6 = 254.4# (LRFD)
Ry = 4.006 kips = 4006 x 1.6 = 6409.6# (LRFD)
Rz = 2.669 kips = 2669# x 1.6 = 4270.4# (LRFD)
Mz = 2.868 k-ft= 34416 #.in x 1.6 = 55065.6 #.in (LRFD)

load processing:
(See next sheet printout)
Connection to Beam

Use (4) Ø1/2" screw bolt+ by dewalt w/4-1/4" min. embedment into concrete, 7" min spacing & 6" min. edge distance.

Stringer Conncection

3/16/2026

1.Project Information

Company: Eastern Engineering Group
Project Engineer: Maria Peroza
Address: Florida 33122
Phone: 305-599-8133
Email: maria.e@easterneg.com
Project Name: Untitled
Project Address: Untitled
Notes:

Connection 3 - Connection to beam

2.Selected Anchor Information

Selected Anchor : Screw-Bolt+
Brand: DEWALT
Material: 1/2" Ø Low Carbon Steel
Embedment: h_{ef} 3.23 in h_{nom} 4.25 in
Approval: ICC-ES ESR-3889
Issued/Revision: Nov,2024 -
Drill method: Hammer Drilled

3.Design Principles

Design Method: ACI 318-19
Load Combinations: Section 5.3 User Defined Loads

4.Base Material Information

Concrete:

Type:	Cracked	Normal Weight Concrete		
Strength	5000 psi			

Reinforcement:

Edge Reinforcement	None or < #4 Rebar			
Spacing	Tension	No (Condition B)	Shear	No (Condition B)
Controls Breakout	Tension	False	Shear	False

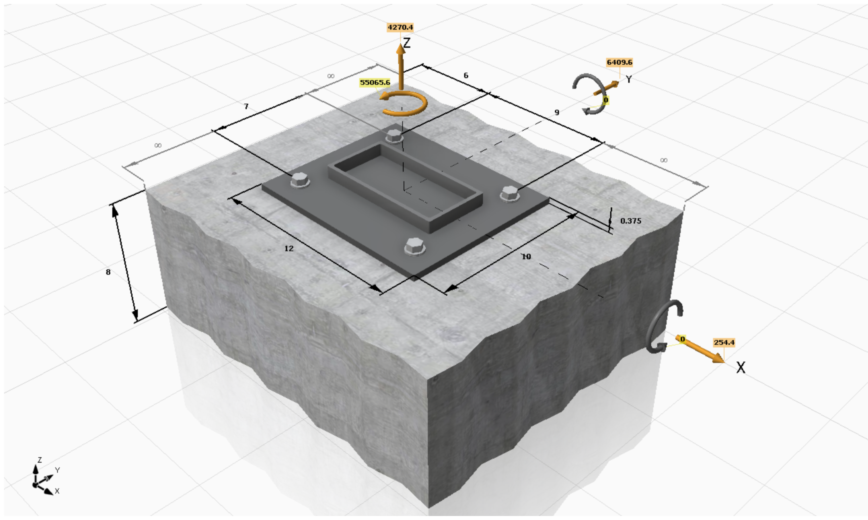
Base Plate:

Sizing	Thickness	0.375	in	Length	10	in	Width 12	in
Standoff	None			Height	0	in		
Strength	36000	psi						

Profile: HSS Rectangular 8 X 4 X 0.250

Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

5.Geometric Conditions



h_{slab}	8	in	h_{min}	6.500	in
Edge Cx-	6	in	c_{min}	1.750	in
Edge Cx+	∞	in	c_{ac}	8.100	in
Edge Cy-	∞	in	s_{min}	2.750	in
Edge Cy+	∞	in			

6.Summary Results

Tension Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	1067.60	13309.00	0.080	OK	
Concrete Breakout Strength:	4270.00	15068.00	0.283	OK	Controls
Pullout Strength:	1068.00	4321.00	0.247	OK	

Shear Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	2415.00	6705.00	0.360	OK	
Concrete Breakout Strength:	4090.00	6847.00	0.597	OK	Controls
Pryout Strength	2415.00	9769.00	0.247	OK	

7.Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

- The results of the calculations carried out by means of the DDA 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 design professional/engineer, particularly with regard to compliance with applicable standards, norms and permits, prior to using them for your specific project. The DDA Software serves only as an aid to interpret standards, 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.
- Verification of a sufficiently stiff base plate in respect to base plate deformations is not performed by the DDA Software, this check should be done by a design professional/engineer outside of the DDA Software. The DDA base plate thickness analysis is based on a proof of stresses, the base plate is assumed to not deform under load actions.

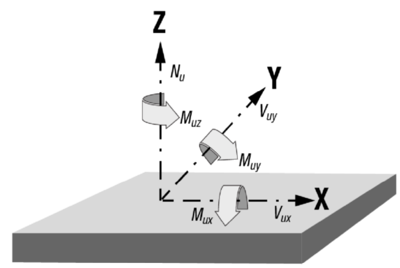
8.Load Condition

Design Loads / Actions

Nu	4270.4	lb	Vux	0.0	lb	Vuy	0.0	lb
Muz	55065.6	in-lb	Mux	0.0	in-lb	Muy	0.0	in-lb

Consider Load Reversal

X Direction 0% Y Direction 0%



Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

9.Load Distribution

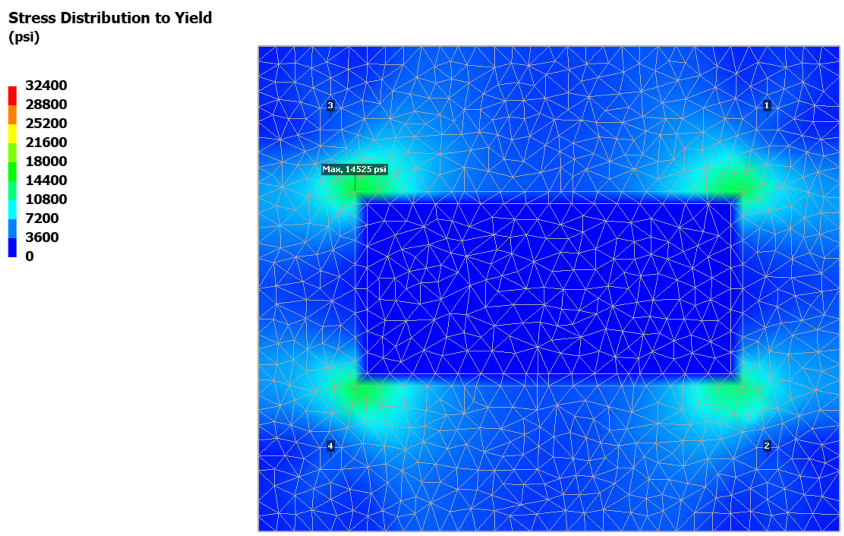
Max. concrete compressive strain:	0.000	%	Anchor Eccentricity					
Max. concrete compressive stress:	0.000	psi	ex	0	in	ey	0	in
Resulting tension force:	4270.400	lb	Profile Eccentricity					
Resulting compression force:	0.000	lb	ex	0	in	ey	0	in

Resulting anchor forces / Load distribution

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb)		Anchor Coordinates (in)	
			Shear X	Shear Y	X	Y
1	1067.60	2414.8	-1482.5	1906.1	4.500	3.500
2	1067.60	2414.8	1482.5	1906.1	4.500	-3.500
3	1067.60	2414.8	-1482.5	-1906.1	-4.500	3.500
4	1067.60	2414.8	1482.5	-1906.1	-4.500	-3.500

10.Base Plate Analysis

Stress Distribution



Details

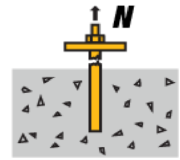
Material		A36		
Yield Stress	f_y	36000		psi
Elastic Modulus	E	29000000		psi
Poisson's Ratio	ν	0.27		
Safety Factor	γ	0.9		
Profile Type		HSS Rectangular	8 X 4 X 0.250	
Max Stress	Σ	14525		psi
Base Plate Thickness	t_p	0.375		in

Base Plate Utilization

$\Sigma / (f_y * \gamma)$	44.8%

11.Design Proof Tension Loading

Steel Strength



ACI 318-19 17.6.1

Variables

N_{sa} (lb)	φ
20475.000	0.65

Results

φN_{sa}	=	13309.0	lb	Table 17.5.2
N_{ua}	=	1067.6	lb	
Utilization	=	8.0%		

Concrete Breakout Strength

ACI 318-19 17.6.2

Equations

$N_{cbg} = (A_{Nc} / A_{Nc0}) . \Psi_{ec,N} . \Psi_{ed,N} . \Psi_{c,N} . \Psi_{cp,N} . N_b$ Eqn. 17.6.2.1b

$N_b = k_c . \lambda_a . (f'_c)^{0.5} . h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
311.936	93.896	1.000	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
8.100	17.000	1.000	5000	3.23	6

N_b (lb)	φ
6978.110	0.65

Results

φN_{cbg}	=	15068	lb	Table 17.5.2
N_{ua}	=	4270.0	lb	
Utilization	=	28.3%		

Pullout Strength:

ACI 318-19 17.6.3

Equations

$N_{pn} = \Psi_{c,P} \cdot N_p \cdot (f'_c / 2500)^n$ Eqn. 17.6.3.1

Variables

$\Psi_{c,P}$	$N_{p,cr}$ (lb)	f'_c (psi)	n	φ
1.000	6647.000	5000	0.500	0.65

Results

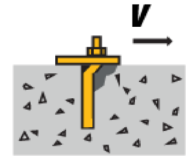
φN_{pn}	=	4321	lb
N_{ua}	=	1068	lb
Utilization	=	24.7%	

Table 17.5.2

12.Design Proof Shear Loading

Reference

Steel Strength



ACI 318-19 17.7.1

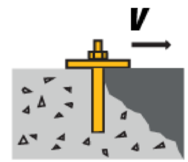
Variables

V_{sa} (lb)	φ
11175.00	0.60

Results

φV_{sa}	=	6705	lb	
V_{ua}	=	2415	lb	Table 17.5.2
Utilization	=	36.0%		

Concrete Breakout Strength



ACI 318-19 17.7.2

Equations

$$V_{cbg} = (A_{Vc} / A_{Vc0}) \cdot \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,v} \cdot V_b$$ Eqn. 17.7.2.1b

$$V_b = (7 \cdot (l_e / d_a)^{0.2} \cdot (d_a)^{0.5}) \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot (c_{a1})^{1.5}$$ Eqn. 17.7.2.2.1a

Variables

A_{Vc} (in²)	A_{Vc0} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
200.000	162.000	1.000	1.000	1.000	1.061

$\Psi_{\alpha,V}$	l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)
1.000	3.230	0.500	1.000	5000.000	6.000

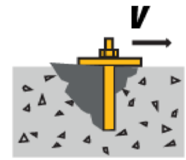
V_b (lb)	φ	V_{cbg} (lb)
7470.358	0.70	9782.113

Results

φV_{cbg}	=	6847	lb	
Direction	=	X-		
V_{ua}	=	4090	lb	Table 17.5.2
Utilization	=	59.7%		

Shear Breakout Case 1 Controls

Pryout Strength
ACI 318-19 17.7.3



Equations

$V_{cpg} = k_{cp} \cdot N_{cpg}$ Eqn. 17.7.3.1b

$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$ Eqn. 17.6.2.1b

$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
93.896	93.896	1.000	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
8.100	17.000	1.000	3.23	5000.000	6.000

N_b (lb)	k_{cp}	N_{cpg} (lb)	φ
6978.108	2.000	6978.108	0.70

Results

φV_{cpg}	=	9769.00	lb	Table 17.5.2
V_{ua}	=	2415	lb	
Utilization	=	24.7%		

13.Interaction of Tension and Shear Loads

Reference

ACI 318-19 17.8

Equations

$(N_{ua} / \varphi \cdot N_n)^{5/3} + (V_{ua} / \varphi \cdot V_n)^{5/3} <= 1.0$ Eqn. 17.8.3

Variables

$N_{ua} / \varphi \cdot N_n$	$V_{ua} / \varphi \cdot V_n$
0.283	0.597

Results

0.546	≤	1.0
Status	:	OK

ANCHOR DESIGN CRITERIA IS SATISFIED

Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 4.00$ Width of Tube Steel (in)

$d := 8.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 0.0$ Moment (Kips-ft)

$V_{weld} := 4.009$ Shear (Kips)

$P_{weld} := 2.669$ Tension or Compression (Kips)

$T_{weld} := 2.868$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $\qquad F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\dfrac{P_{weld}}{2(b+d)\cdot x} + \dfrac{M_{weld}\cdot 12}{\left(b\cdot d + \dfrac{d^2}{3}\right)\cdot x}\right]^2 + \left[\dfrac{1.5V_{weld}}{2(b+d)\cdot x} + \dfrac{T_{weld}\cdot 12\cdot\sqrt{\left(\dfrac{b}{2}\right)^2 + \left(\dfrac{d}{2}\right)^2}}{\dfrac{(b+d)^3}{6}\cdot x}\right]^2} \geq F_v \\ \quad x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.04$ in

$S_{weld} := \left(b\cdot d + \dfrac{d^2}{3}\right)\cdot t_e$ $\qquad S_{weld} = 2.13$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $\qquad A_{weld} = 0.96$ in^2

$J_{weld} := \dfrac{(b+d)^3}{6}\cdot t_e$ $\qquad J_{weld} = 11.52$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$ $f_b = 0.00$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$ $f_v = 19.62$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$ $f_a = 2.78$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^{\,2}}$$ $f_{weld} = 19.82$ ksi

$$\text{STRESS} := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{cases}$$ STRESS = "OK"

$$W_{weld} := \begin{cases} \text{"3/16"} & \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} & \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} & \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} & \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} & \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} & \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} & \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{cases}$$

$W_{weld} = \text{"3/16"}$

Eastern Engineering Group
3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

Connection 4

loads:
N14
Rx = 0.25 kips = 250 # x 1.6 = 400# (LRFD)
Ry = 5.022 kips = 5022 x 1.6 = 8035.2# (LRFD)
Rz = 3.249 kips = 3249# x 1.6 = 5198.4# (LRFD)
Mx = 5.874 k-ft= 70488 #.in x 1.6 = 112780.8 #.in (LRFD)
My = 1.09 k-ft = 13080 #.in x 1.6 = 20928 #.in (LRFD)
Mz = 0.97 k-ft = 11640 #.in x 1.6 =18624 #.in (LRFD)

load processing:
(See next sheet printout)
Connection to Slab

Use (6) Ø1/2" screw bolt+ by dewalt w/4-1/4" min. embedment into concrete, 8" min spacing & 8" min. edge distance.

Stringer Connection to Slab

3/16/2026

1.Project Information

Company:	Eastern Engineering Group
Project Engineer:	Maria Peroza
Address:	Florida 33122
Phone:	305-599-8133
Email:	maria.e@easterneg.com
Project Name:	Untitled
Project Address:	Untitled
Notes:	

Connection 4 - Connection to Slab

2.Selected Anchor Information

Selected Anchor :	Screw-Bolt+
Brand:	DEWALT
Material:	1/2" Ø Low Carbon Steel
Embedment:	h_{ef} 3.23 in h_{nom} 4.25 in
Approval:	ICC-ES ESR-3889
Issued/Revision:	Nov,2024 -
Drill method:	Hammer Drilled

3.Design Principles

Design Method:	ACI 318-19	
Load Combinations:	Section 5.3	User Defined Loads

4.Base Material Information

Concrete:

Type:	Cracked	Normal Weight Concrete		
Strength	5000 psi			

Reinforcement:

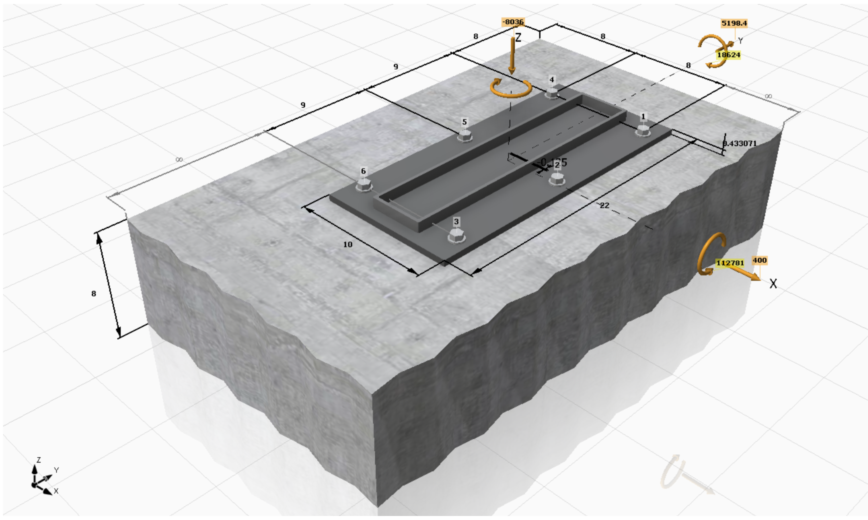
Edge Reinforcement	None or < #4 Rebar			
Spacing	Tension	No (Condition B)	Shear	No (Condition B)
Controls Breakout	Tension	False	Shear	False

Base Plate:

Sizing	Thickness	0.4330 71	in	Length	22	in	Width	10 in
Standoff	None			Height	0	in		
Strength	36000	psi						

Profile: HSS Rectangular 20 X 4 X 0.250

5.Geometric Conditions



h_{slab}	8	in	h_{min}	6.500	in
Edge Cx-	8	in	c_{min}	1.750	in
Edge Cx+	∞	in	c_{ac}	8.100	in
Edge Cy-	∞	in	s_{min}	2.750	in
Edge Cy+	8	in			

6.Summary Results

Tension Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	1193.01	13309.00	0.090	OK	
Concrete Breakout Strength:	2813.00	9364.00	0.300	OK	Controls
Pullout Strength:	1193.00	4321.00	0.276	OK	

Shear Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	1183.00	6705.00	0.176	OK	
Concrete Breakout Strength:	5254.00	10956.00	0.480	OK	Controls
Pryout Strength	5663.00	24975.00	0.227	OK	

7.Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

- The results of the calculations carried out by means of the DDA 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 design professional/engineer, particularly with regard to compliance with applicable standards, norms and permits, prior to using them for your specific project. The DDA Software serves only as an aid to interpret standards, 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.
- Verification of a sufficiently stiff base plate in respect to base plate deformations is not performed by the DDA Software, this check should be done by a design professional/engineer outside of the DDA Software. The DDA base plate thickness analysis is based on a proof of stresses, the base plate is assumed to not deform under load actions.

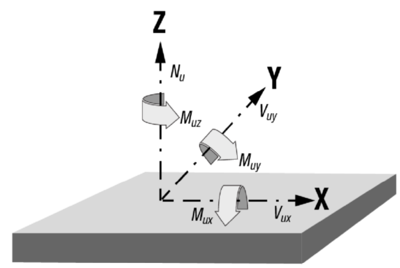
8.Load Condition

Design Loads / Actions

Nu	-8036.0	lb	Vux	400.0	lb	Vuy	5198.4	lb
Muz	-20928.0	in-lb	Mux	112781.0	in-lb	Muy	18624.0	in-lb

Consider Load Reversal

X Direction 0% Y Direction 0%



9.Load Distribution

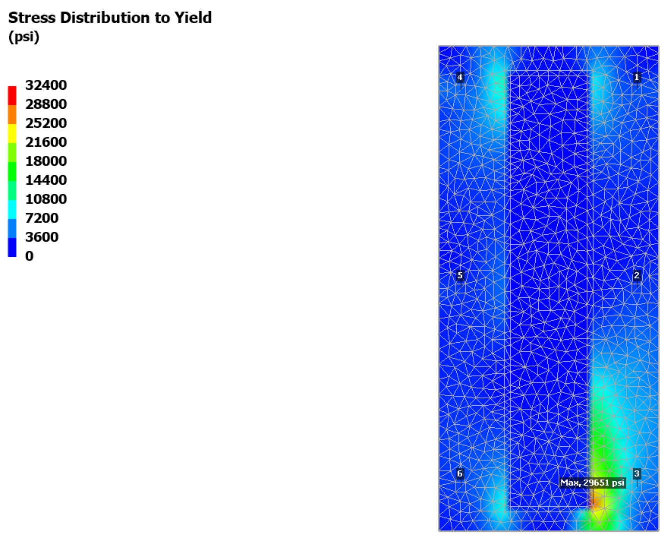
Max. concrete compressive strain:	0.117	%	Anchor Eccentricity				
Max. concrete compressive stress:	509.847	psi	ex	0	in	ey 0	in
Resulting tension force:	2812.569	lb	Profile Eccentricity				
Resulting compression force:	10848.570	lb	ex	0	in	ey -0.125	in

Resulting anchor forces / Load distribution

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb) Shear X	Component Shear Load (lb) Shear Y	Anchor Coordinates (in) X	Anchor Coordinates (in) Y
1	868.52	842.5	514.1	667.6	4.000	9.000
2	213.28	670.9	66.7	667.6	4.000	0.000
3	0.00	768.5	-380.7	667.6	4.000	-9.000
4	1193.01	1182.8	514.1	1065.2	-4.000	9.000
5	537.77	1067.3	66.7	1065.2	-4.000	0.000
6	0.00	1131.2	-380.7	1065.2	-4.000	-9.000

10.Base Plate Analysis

Stress Distribution



Details

Material		A36	
Yield Stress	f_y	36000	psi
Elastic Modulus	E	29000000	psi
Poisson's Ratio	ν	0.27	
Safety Factor	γ	0.9	
Profile Type		HSS Rectangular 20 X 4 X 0.250	
Max Stress	Σ	29651	psi
Base Plate Thickness	t_p	0.433	in

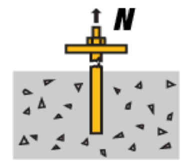
Base Plate Utilization

$\Sigma / (f_y * \gamma)$	91.5%

11.Design Proof Tension Loading

Steel Strength

ACI 318-19 17.6.1



Variables

N_{sa} (lb)	φ
20475.000	0.65

Results

φN_{sa}	=	13309.0	lb	Table 17.5.2
N_{ua}	=	1193.0	lb	
Utilization	=	9.0%		

Concrete Breakout Strength

ACI 318-19 17.6.2

Equations

$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$ Eqn. 17.6.2.1b

$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
330.626	93.896	0.586	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
8.100	17.000	1.000	5000	3.23	8

N_b (lb)	φ
6978.110	0.65

Results

φN_{cbg}	=	9364	lb	Table 17.5.2
N_{ua}	=	2813.0	lb	
Utilization	=	30.0%		

Pullout Strength:

ACI 318-19 17.6.3

Equations

$N_{pn} = \Psi_{c,P} \cdot N_p \cdot (f'_c / 2500)^n$ Eqn. 17.6.3.1

Variables

$\Psi_{c,P}$	$N_{p,cr}$ (lb)	f'_c (psi)	n	φ
1.000	6647.000	5000	0.500	0.65

Results

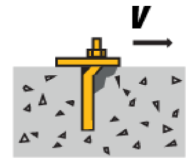
φN_{pn}	=	4321	lb	
N_{ua}	=	1193	lb	Table 17.5.2
Utilization	=	27.6%		

12.Design Proof Shear Loading

Reference

Steel Strength

ACI 318-19 17.7.1



Variables

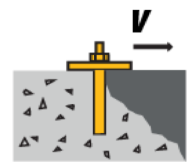
V_{sa} (lb)	φ
11175.00	0.60

Results

φV_{sa}	=	6705	lb	
V_{ua}	=	1183	lb	Table 17.5.2
Utilization	=	17.6%		

Concrete Breakout Strength

ACI 318-19 17.7.2



Equations

$$V_{cbg} = (A_{Vc} / A_{Vc0}) \cdot \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,v} \cdot V_b$$

Eqn. 17.7.2.1b

$$V_b = (7 \cdot (l_e / d_a)^{0.2} \cdot (d_a)^{0.5}) \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot (c_{a1})^{1.5}$$

Eqn. 17.7.2.2.1a

Variables

A_{Vc} (in²)	A_{Vc0} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
400.000	1152.000	1.000	0.800	1.000	1.732

$\Psi_{\alpha,V}$	l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)
1.000	3.230	0.500	1.000	5000.000	16.000

V_b (lb)	φ	V_{cbg} (lb)
32530.783	0.70	15651.380

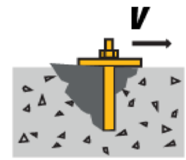
Results

φV_{cbg}	=	10956	lb	
Direction	=	X-		
V_{ua}	=	5254	lb	Table 17.5.2
Utilization	=	48.0%		

Shear Breakout Case 2 Controls

Pryout Strength

ACI 318-19 17.7.3



Equations

$V_{cpg} = k_{cp} \cdot N_{cpg}$ Eqn. 17.7.3.1b

$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$ Eqn. 17.6.2.1b

$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ Eqn. 17.6.2.2.1

Variables

A_{Nc} (in²)	A_{Nc0} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
489.836	93.896	0.490	1.000	1.000	1.000

c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
8.100	17.000	1.000	3.23	5000.000	8.000

N_b (lb)	k_{cp}	N_{cpg} (lb)	φ
6978.108	2.000	17839.601	0.70

Results

φV_{cpg}	=	24975.00	lb	Table 17.5.2
V_{ua}	=	5663	lb	
Utilization	=	22.7%		

13.Interaction of Tension and Shear Loads

Reference

ACI 318-19 17.8

Equations

$(N_{ua} / \varphi \cdot N_n)^{5/3} + (V_{ua} / \varphi \cdot V_n)^{5/3} <= 1.0$ Eqn. 17.8.3

Variables

$N_{ua} / \varphi \cdot N_n$	$V_{ua} / \varphi \cdot V_n$
0.300	0.480

Results

0.428	≤	1.0
Status	:	OK

ANCHOR DESIGN CRITERIA IS SATISFIED

Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 4.00$ Width of Tube Steel (in)

$d := 18.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 5.953$ Moment (Kips-ft)

$V_{weld} := 3.258$ Shear (Kips)

$P_{weld} := 5.022$ Tension or Compression (Kips)

$T_{weld} := 1.09$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $\quad F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\frac{P_{weld}}{2(b+d)\cdot x} + \frac{M_{weld}\cdot 12}{\left(b\cdot d + \frac{d^2}{3}\right)\cdot x}\right]^2 + \left[\frac{1.5V_{weld}}{2(b+d)\cdot x} + \frac{T_{weld}\cdot 12\cdot\sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6}\cdot x}\right]^2} \ge F_v \\ \quad x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.03$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $\quad S_{weld} = 5.40$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $\quad A_{weld} = 1.32$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $\quad J_{weld} = 53.26$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$

$f_b = 13.22$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$

$f_v = 5.97$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$

$f_a = 3.80$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^2}$$

$f_{weld} = 18.04$ ksi

$$STRESS := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{array} \right.$$

$STRESS = \text{"OK"}$

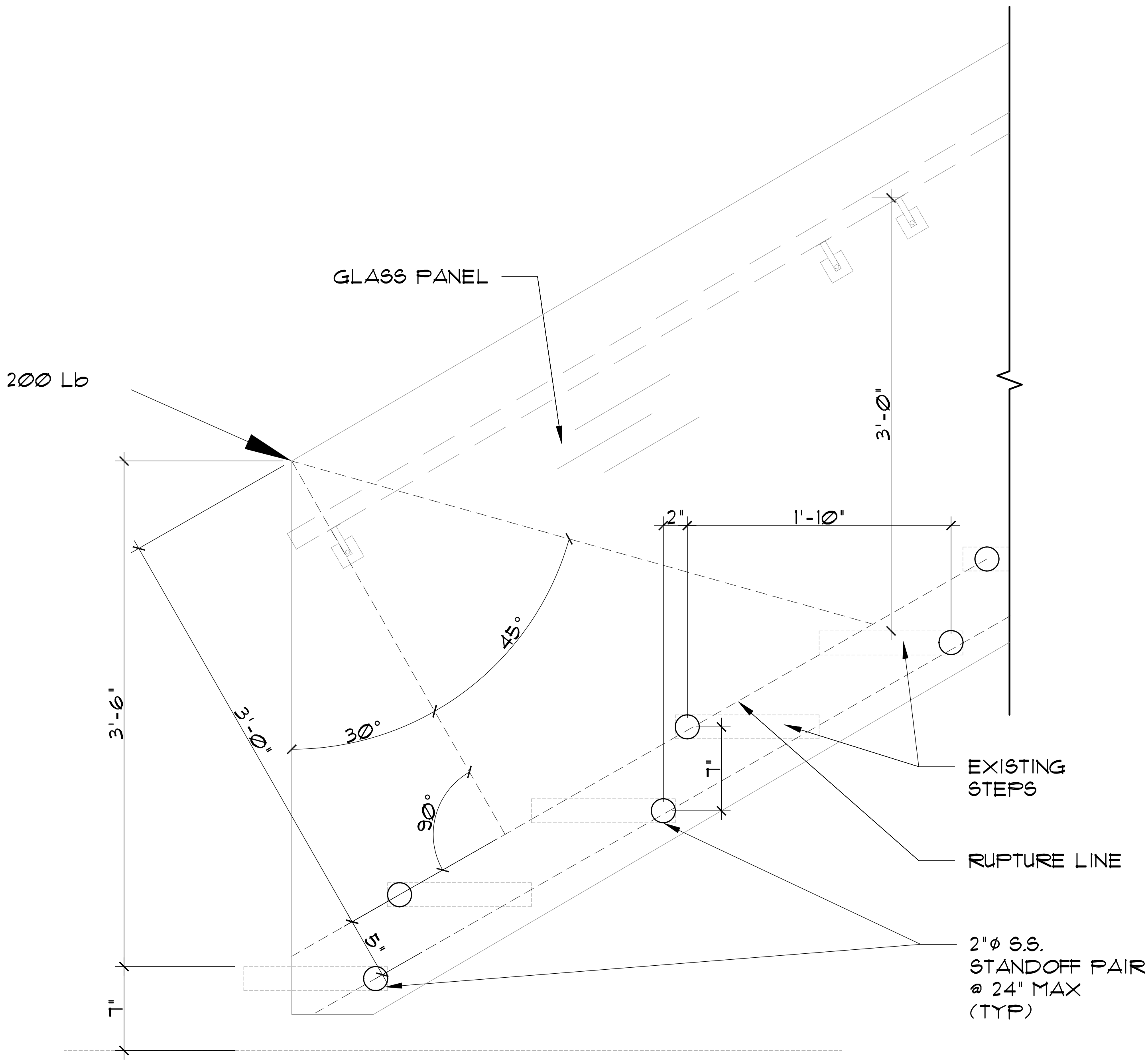
$$W_{weld} := \left| \begin{array}{l} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{array} \right.$$

$W_{weld} = \text{"3/16"}$

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

INTERIOR GLASS RAILING DESIGN

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
Phone: (305) 599-8133
info@easterneg.com
www.easterneg.com



**RIGID PANEL DISTRIBUTES LOAD EVENLY BETWEEN BOTH STANDOFF PAIR

CONSERVATIVE ANALYSIS

Min 2 Standoff pairs by glass panel

Glass Effective Laminated Thickness

Laminate Glass Data:

$E := 10400ksi$ Young Modulus of Elasticity of Glass E MPa
For Soda Lime Glass: E=10 400 Ksi (71705.5 MPa)

$h_v := 0.06in$ Interlayer Thickness (in)

$h_1 := 0.219 \cdot in$ Glass Ply 1 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$h_2 := 0.219 \cdot in$ Glass Ply 2 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$a := 34in$ Length Scale (Smallest in-Plane Dimension of the Laminate Plate (in)

$G := 3830psi$ Interlayer Storage Shear Modulus G=3830 psi (For Wind 3s/50 Celcius)
For Exterior: SentryGlass 1,640 psi (11.3 MPa) PVB 70 psi (0.48 MPa)
For Interior: SentryGlass 3830 psi (26.4 MPa) PVB 140 psi (0.96 MPa)

Then

$$h_s := 0.5 \cdot (h_1 + h_2) + h_v \qquad h_s = 0.279 \cdot in$$

$$h_{s.1} := \frac{h_s \cdot h_1}{h_1 + h_2} \qquad h_{s.1} = 0.140 \cdot in$$

$$h_{s.2} := \frac{h_s \cdot h_2}{h_1 + h_2} \qquad h_{s.2} = 0.140 \cdot in$$

$$I_s := h_1 \cdot h_{s.2}^{\;2} + h_2 \cdot h_{s.2}^{\;2} \qquad I_s = 0.009 \cdot in \cdot in \cdot in$$

$$\Gamma := \frac{1}{1 + \dfrac{9.6 \cdot E \cdot I_s \cdot h_v}{G \cdot h_s^{\;2} \cdot a^2}} \qquad \Gamma = 0.871$$

Laminate Effective Thickness For Deflection

$$h_{efw} := \sqrt[3]{h_1^{\;3} + h_2^{\;3} + 12 \cdot \Gamma \cdot I_s} \qquad h_{efw} = 0.479 \cdot in$$

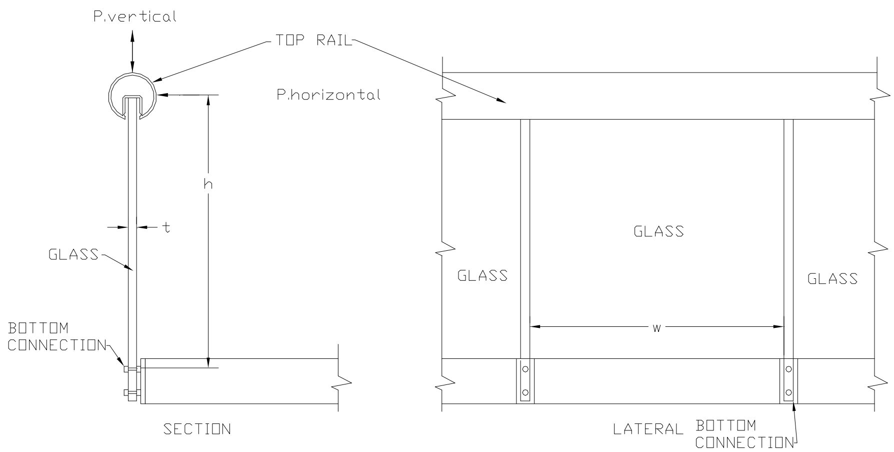
Laminate Effective Thickness For Bending Stress

$$h_{1.ef\sigma} := \sqrt{\frac{h_{efw}^{\;3}}{\max(h_1, h_2) + 2 \cdot \Gamma \cdot \max(h_{s.1}, h_{s.2})}} \qquad h_{1.ef\sigma} = 0.488 \cdot in$$

TABLE 4 Nominal and Minimum Glass Thicknesses

Nominal Thickness or Designation, mm (in.)	Minimum Thickness, mm (in.)
2.5 (3⁄32)	2.16 (0.085)
2.7 (lami)	2.59 (0.102)
3.0 (1⁄8)	2.92 (0.115)
4.0 (5⁄32)	3.78 (0.149)
5.0 (3⁄16)	4.57 (0.180)
6.0 (1⁄4)	5.56 (0.219)
8.0 (5⁄16)	7.42 (0.292)
10.0 (3⁄8)	9.02 (0.355)
12.0 (1⁄2)	11.91 (0.469)
16.0 (5⁄8)	15.09 (0.595)
19.0 (3⁄4)	18.26 (0.719)
22.0 (7⁄8)	21.44 (0.844)
25.0 (1)	24.61 (0.969)

Glass Railing Design w/ Standoff Support



Loads Data:

$P_{200} := 200.00$ Concentrated Load (lbs)

$q_{50} := 50.00$ Uniform Load (plf)

$q_{wind} := 5.00$ Uniform Distributed Load (psf)

Glass Data:

$E := 10400000.00$ Modulus of Elasticity of Glass (psi)

$Mr_{flexure} := 24000.00$ Modulus of Rupture of Glass in Flexure For Live Loads (psi)

$F_{b.wind} := 9600.00$ Allowable Stress in Flexure For Wind Loads Per ASTM E1300 (psi)

$Mr_{shear} := 12000.00$ Modulus of Rigity of glass in Shear (psi)

$SF := 4.00$ Safety Factor

Geometric Glass Railing Data:

$h := 38.0$ Height of Glass Lite Pannel in Cantilever (in)

$t_s := 0.488$ Effective Thickness of Glass Pannel for Stress (in) Per ASTM E1300 Appendix X9

$t_d := 0.479$ Effective Thickness of Glass Pannel for Deflection (in) Per ASTM E1300 Appendix X9

$w_{min} := 34.00$ **Minimum** Width of Glass Pannel (in)

$w_{max} := 54.00$ **Maximum** Width of Glass Pannel (in)

Then

$$F_{b.live} := \frac{Mr_{flexure}}{SF}$$ $F_{b.live} = 6000.00$ psi

$$F_v := \frac{Mr_{shear}}{SF}$$ $F_v = 3000.00$ psi

$$L := \begin{vmatrix} h \\ w_{min} \text{ if } w_{min} < h \end{vmatrix}$$ $L = 34.00$ in

$$S_x := \frac{L \cdot t_s^{\,2}}{6}$$ $S_x = 1.35$ in^3

$$I_x := \frac{L \cdot t_s^{\,3}}{12}$$ $I_x = 0.33$ in^4

$$A := L \cdot t_s$$ $A = 16.59$ in^2

Actual Glass Moment in Length (L):

Concentrated Load = 200 lbs.

$$M_{200} := P_{200} \cdot h$$ $M_{200} = 7600.00$ lbs – in

Uniform Load = 50 plf

$$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$$ $M_{50} = 5383.33$ lbs – in

Uniform Distributed Wind Load

$$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$$ $M_{wind} = 852.36$ lbs – in

$$M_{Glass.live} := \max(M_{200}, M_{50})$$ $M_{Glass.live} = 7600.00$ lbs – in

$$M_{Glass.wind} := M_{wind}$$ $M_{Glass.wind} = 852.36$ lbs – in

$$M_{Glass} := \max(M_{Glass.live}, M_{Glass.wind})$$ $M_{Glass} = 7600.00$ lbs – in

Actual Glass Shear in Length (L):

Concentrated Load = 200 lbs.

$$V_{200} := P_{200}$$ $V_{200} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50} := q_{50} \cdot \frac{L}{12}$$ $V_{50} = 141.67$ lbs

Uniform Distributed Wind Load

$$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$$ $V_{wind} = 44.86$ lbs

$$V_{Glass} := \max(V_{200}, V_{50}, V_{wind})$$ $V_{Glass} = 200.00$ lbs

Section Required:

Bending Design:
Section Modulus Required

$$Sx_r := \max\left(\frac{M_{Glass.live}}{F_{b.live}}, \frac{M_{Glass.wind}}{F_{b.wind}}\right)$$ $Sx_r = 1.27$ in^3

Shear Design:
Area Required

$$A_r := \frac{V_{Glass}}{F_v}$$ $A_r = 0.07$ in^2

Section Provided:

$$BENDING_{glass} := \text{if}\left(Sx_r \geq \min(S_x), \text{"N.G"}, \text{"OK"}\right)$$ $BENDING_{glass} = \text{"OK"}$

$$SHEAR_{glass} := \text{if}\left(A_r \geq A, \text{"N.G"}, \text{"OK"}\right)$$ $SHEAR_{glass} = \text{"OK"}$

Check Deflection:

$$\Delta_{allow} := \frac{h}{30}$$ $\Delta_{allow} = 1.27$ in

$$I_d := \frac{L \cdot t_d^{\,3}}{12}$$ $I_d = 0.31$ in^4

$$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_d}$$ $\Delta_{200} = 1.13$ in

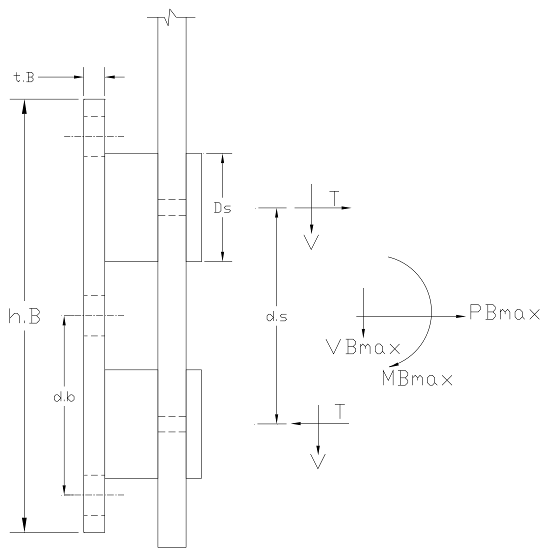
$$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_d}$$ $\Delta_{wind} = 0.10$ in

$$DEFLECTION_{glass} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} \quad \text{if } \max\left(\Delta_{200}, \Delta_{wind}\right) \le \Delta_{allow} \end{cases}$$

$DEFLECTION_{glass} = \text{"O.K."}$

Base Connection Design:

Stainless Steel



$d_s := 4.00$ Distance Between Glass Supports in Connection (in)

$D_s := 2.00$ Diameter of Glass Support (in)

$d_b := 4.00$ Distance Between Anchors Bolts (in)

$P_{system} := 120.00$ Pannel Total Weigth of Glass and Handrail (lbs)

Maximum Connection Moment:

$$M_{Bactual} := \frac{M_{Glass}}{2}$$

$\boxed{M_{Bactual} = 3800.00}$ lbs – in

Maximum Connection Shear:

$$V_{Bactual} := \frac{V_{Glass}}{2} + \frac{P_{system}}{2}$$

$\boxed{V_{Bactual} = 160.00}$ lbs

Maximum Connection Axial:

$$P_{Bactual} := \frac{V_{Glass}}{2}$$

$\boxed{P_{Bactual} = 100.00}$ lbs

Tensile Load Per Glass Supports in Connection:

$$T := \frac{M_{Bactual}}{d_s} + P_{Bactual}$$

$\boxed{T = 1050.00}$ lbs

Shear Load Per Glass Supports in Connection:

$$V := \frac{\max(V_{200}, V_{50})}{2} + \frac{P_{system}}{2}$$

$\boxed{V = 160.00}$ lbs

Check Two Way Glass Shear at Glass Supports in Connection:

$$A_p := \pi \cdot (D_s + t_s)$$

$\boxed{A_p = 7.82}$ in^2

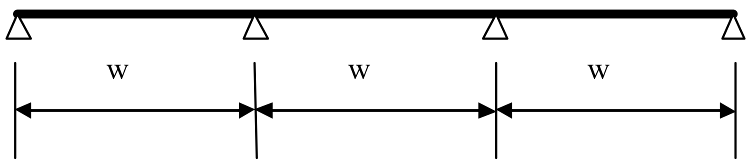
$$f_v := \frac{T}{A_p \cdot t_s}$$

$\boxed{f_v = 275.28}$ psi

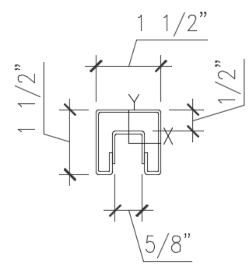
$$SHEAR_{twoway} := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq F_v \end{array} \right.$$

$\boxed{SHEAR_{twoway} = \text{"OK"}}$

Top Railing Design:



Handrail to provide redistribution of load between glass panels & to remain in place in case that one of the glass planes breaks



TOP RAIL ($1\frac{1}{2}$"x$1\frac{1}{2}$"x$\frac{1}{16}$" THK)

Area = 0.5576 in^2
Ixx = 0.1191 in^4
Iyy = 0.1428 in^4
Jz = 0.2619 in^4
Pxy = 0.0000 in^4
rxx = 0.4621 in
ryy = 0.5061 in
Sxx max = 0.1659 in^3
Sxx min = 0.1522 in^3
Syy max = 0.1905 in^3
Syy min = 0.1905 in^3

$F_{b.top} := 15000.00$ (Conserv.) Top Railing Allowable Bending Stress (psi)

$F_{v.top} := 8982.00$ (Conserv.) Top Railing Allowable Shear Stress (psi)

$Sx_{top} := 0.1522$ Top Railing Inertia Modulus for Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.1905$ Top Railing Inertia Modulus for Horizontal Loads $\left(in^3\right)$

$A_{top} := 0.5576$ Top Railing Area $\left(in^2\right)$

Actual Moment:

Concentrated Load = 200 lbs.

$$M_{200.top} := \frac{P_{200} \cdot w_{max}}{5} \qquad M_{200.top} = 2160.00 \quad lb - in$$

Uniform Load = 50 plf

$$M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot w_{max}^{\ 2} \qquad M_{50.top} = 1229.58 \quad lb - in$$

$$M_{actual.top} := \max\left(M_{200.top}, M_{50.top}\right) \qquad M_{actual.top} = 2160.00 \quad lb - in$$

Actual Shear:

Concentrated Load = 200 lbs.

$$V_{200.top} := P_{200}$$

$V_{200.top} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot w_{max}$$

$V_{50.top} = 135.00$ lbs

$$V_{actual.top} := \max\left(V_{200.top}, V_{50.top}\right)$$

$V_{actual.top} = 200.00$ lbs

Section Required:

Bending Design:

Section Modulus Required

$$Shr := \frac{M_{actual.top}}{F_{b.top}}$$

$Shr = 0.14$ in^3

Shear Design:

Area Required

$$Ahr := \frac{1.5 \cdot V_{actual.top}}{F_{v.top}}$$

$Ahr = 0.03$ in^2

Section Provided:

$$BENDING_{top} := \text{if}\left(Shr \geq \min\left(Sx_{top}, Sy_{top}\right), \text{"N.G"}, \text{"OK"}\right)$$

$BENDING_{top} = \text{"OK"}$

$$SHEAR_{top} := \text{if}\left(Ahr \geq A_{top}, \text{"N.G"}, \text{"OK"}\right)$$

$SHEAR_{top} = \text{"OK"}$

Steel Bolts Threaded Moment &Tension & Shear Design 16th

Data:

BOLT A307

$M_{bolt} := 0.0$ Moment in Anchor (kips) $\Omega := 2.00$ ASD Factor=2.00

$T_{bolt} := 1.43$ Tension in Anchor (kips) (Conservative)

$V_{bolt} := 0.20$ Shear in Anchor (kips) (Conservative)

$F_{nv} := 27.00$ Nominal Shear Strength (ksi) Table J3.2 AISC 16.1-120 ....=0.45*Fu

$F_{nt} := 45.00$ Nominal Tensile Strength (ksi) Table J3.2 AISC 16.1-120......=0.75*Fu

$D := 0.375$ Screw Diameter (in) $n := 16.0$ Threads per in......... (AISC-Table 7-17 page 7-81)

$$A_{bolt} := 0.7854 \cdot \left(D - \frac{0.9743}{n}\right)^2 \qquad A_{bolt} = 0.077 \ \text{in}^2$$

$$S_{bolt} := 0.098 \left(D - \frac{0.9743}{n}\right)^3 \qquad S_{bolt} = 0.00 \ \text{in}^3$$

Check Shear

$$f_v := \frac{V_{bolt}}{A_{bolt}} \qquad f_v = 2.58 \ \text{ksi}$$

$$V_{allow} := \frac{F_{nv} \cdot A_{bolt}}{\Omega} \qquad V_{allow} = 1.05 \ \text{kips}$$

$$SHEAR := \text{if}\left(V_{bolt} \le V_{allow}, \text{"OK"}, \text{"NG"}\right) \qquad SHEAR = \text{"OK"}$$

Check Tension

$$f_t := \frac{T_{bolt}}{A_{bolt}} + \frac{M_{bolt}}{S_{bolt}} \qquad f_t = 18.45 \ \text{ksi}$$

$$F'_{nt} := \min\left(F_{nt}, \max\left(0, 1.3F_{nt} - \Omega \cdot \frac{F_{nt}}{F_{nv}} \cdot f_v\right)\right) \qquad F'_{nt} = 45.00 \ \text{ksi}$$

$$T_{allow} := \frac{F'_{nt} \cdot A_{bolt}}{\Omega} \qquad T_{allow} = 1.74 \ \text{kips}$$

$$TENSION := \text{if}\left(T_{bolt} \le T_{allow}, \text{"OK"}, \text{"NG"}\right) \qquad TENSION = \text{"OK"}$$

Minimum Thread Length

$$TL_{min} := \frac{2 \cdot T_{bolt}}{\frac{F_{nv}}{\Omega} \cdot \pi \cdot \left(D - \frac{0.9743}{n}\right)} \qquad TL_{min} = 0.21 \ \text{in}$$

Eastern Engineering Group
3401 NW 82nd Ave.
Suite 370
Doral, FL. 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

Minimum Thread Length Design

.

*for 3/8" Diam Bolt A307 Ttable= Available Tensile Strength * Gross Bolt Area=2.475kips

.

Available Tensile Strength =22.5kips (see attached table 7-2)
Gross Bolt Area=0.11 (see attached table 7-17)

.

Fu=60ksi (table 2-6 see attached)
Ltable=0.25" Nut Thread Length for Non High-Strength Bolts (see attached table 7-20)

Minimum Base Thickness for Bolt Connection Design AISC

Load Data:

$T_{bolt} := 1.472$ Tension in Anchor (kips) conservative

Bolt Data:

$D := 0.375$ Bolt Diameter (in)

$n := 16.00$ Threads per in......... (AISC-ASD Table 4-147)

Base Material Data:

$Fv_{base} := 18.40$ Base Material Allowable Shear Strength (ksi)

$Fu_{base} := 58.00$ Base Material Tensile Strength (ksi) by Table 2-3 or 2-4 page 2-39 or 2-40

AISC Equivalent Bolts Thread Length Data:

$T_{table} := 2.475$ Maximum Allowable Tension in Anchor by Table 7-2 page 7-23

$Fu_{table} := 60.00$ Maximimun Tensile Strength (ksi) by Table 2-5 page 2-41

$L_{table} := 0.25$ Nut Thread Length (in) For High-Strength Bolts..............Table 7-15 page 7-80
For Non High-Strength Bolts.......Table 7-18 page 7-83

Minimum Thread Length (by Calculations)

$$TL_{calc} := \frac{2 \cdot T_{bolt}}{Fv_{base} \cdot \pi \cdot \left(D - \frac{0.9743}{n}\right)}$$

$TL_{calc} = 0.162$ in

Minimum Thread Length (by Equivalent AISC Standard)

$$TL_{equivalent} := L_{table} \cdot \frac{T_{bolt}}{T_{table}} \cdot \frac{Fu_{table}}{Fu_{base}}$$

$TL_{equivalent} = 0.154$ in

Minimum Thread Length

$$TL_{min} := \max\left(TL_{calc}, TL_{equivalent}\right)$$

$TL_{min} = 0.162$ in

USE MINIMUM 1/4"
CONSERVATIVE

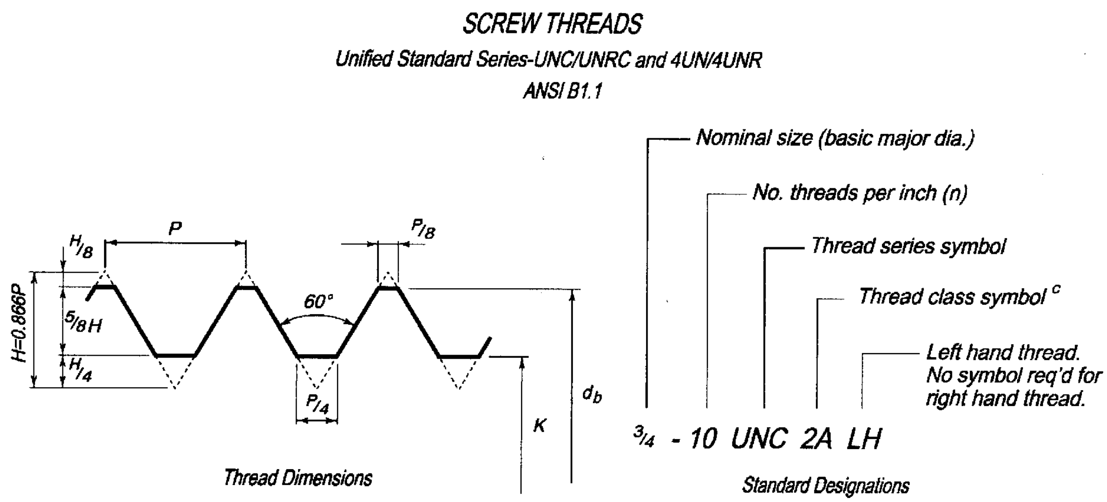
Table 7–2
Available Tensile Strength of Bolts, kips

Nominal Bolt Diameter d_b, in.			5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}			0.307		0.442		0.601		0.785	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	13.8	20.7	19.9	29.8	27.1	40.6	35.3	53.0
A490	56.5	84.8	17.3	26.0	25.0	37.4	34.0	51.0	44.4	66.6
A307	22.5	33.8	6.90	10.4	9.94	14.9	13.5	20.3	17.7	26.5

Nominal Bolt Diameter d_b, in.			1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}			0.994		1.23		1.48		1.77	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	44.7	67.1	55.2	82.8	66.8	100	79.5	119
A490	56.5	84.8	56.2	84.2	69.3	104	83.9	126	99.8	150
A307	22.5	33.8	22.4	33.5	27.6	41.4	33.4	50.1	39.8	59.6

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

Table 7–17
Threading Dimensions for High-Strength and Non-High-Strength Bolts



Diameter		Area			
Bolt Diameter d_b, in.	**Min. Root K, in.**	**Gross Bolt Area, in.^{2}**	**Min. Root Area, in.^{2}**	**Net Tensile Area[a], in.^{2}**	**Threads per inch, n[b]**
1/4	0.196	0.0490	0.0301	0.0320	20
3/8	0.307	0.110	0.0742	0.0780	16
1/2	0.417	0.196	0.136	0.142	13
5/8	0.527	0.307	0.218	0.226	11
3/4	0.642	0.442	0.323	0.334	10
7/8	0.755	0.601	0.447	0.462	9
1	0.865	0.785	0.587	0.606	8
1 1/8	0.970	0.994	0.740	0.763	7
1 1/4	1.10	1.23	0.942	0.969	7
1 3/8	1.19	1.49	1.12	1.16	6
1 1/2	1.32	1.77	1.37	1.41	6
1 3/4	1.53	2.41	1.85	1.90	5
2	1.76	3.14	2.43	2.50	4.5
2 1/4	2.01	3.98	3.17	3.25	4.5
2 1/2	2.23	4.91	3.90	4.00	4
2 3/4	2.48	5.94	4.83	4.93	4
3	2.73	7.07	5.85	5.97	4
3 1/4	2.98	8.30	6.97	7.10	4
3 1/2	3.23	9.62	8.19	8.33	4
3 3/4	3.48	11.0	9.51	9.66	4
4	3.73	12.6	10.9	11.1	4

[a] Net tensile area $= 0.7854 \times \left(d_b - \dfrac{0.9743}{n}\right)^2$

[b] For diameters listed, thread series is UNC (coarse). For larger diameters, thread series is 4UN.

[c] 2A denotes Class 2A fit applicable to external threads;
2B denotes corresponding Class 2B fit for internal threads.

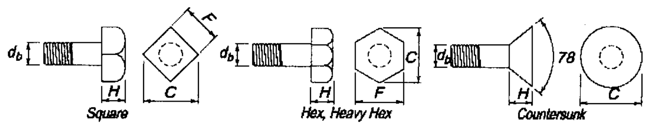
Table 2–6
Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation	F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts: Conventional	High-Strength Bolts: Twist-Off-Type Tension-Control[d]	Common Bolts	Nuts	Washers	Direct-Tension-Indicator Washers	Threaded Rods	Shear Stud Connectors	Anchor Rods: Hooked	Anchor Rods: Headed	Anchor Rods: Threaded & Nutted
A108	—	65	0.375 to 0.75, incl.								■			
A325[d]	—	105	over 1 to 1.5 incl.	■										
	—	120	0.5 to 1, incl.	■										
A490	—	150	0.5 to 1.5	■										
F1852	—	105	1.125		■									
	—	120	0.5 to 1, incl.		■									
A194 Gr. 2H	—	—	0.25 to 4				▒							
A563	—	—	0.25 to 4				■							
F436[b]	—	—	0.25 to 4					■						
F959	—	—	0.5 to 1.5						■					
A36	36	58-80	to 10							■		▒		▒
A193 Gr. B7[e]	—	100	over 4 to 7							▒		▒		▒
	—	115	over 2.5 to 4							▒		▒		▒
	—	125	2.5 and under							▒		▒		▒
A307 Gr. A	—	60	0.25 to 4			■							▒	
A307 Gr. C	—	58-80	0.25 to 4							▒		▒		▒
A354 Gr. BD	—	140	2.5 to 4 incl.							▒		▒	▒	▒
	—	150	0.25 to 2.5, incl.							▒		▒	▒	▒
A449	—	90	1.75 to 3 incl.	c						▒		▒	▒	▒
	—	105	1.125 to 1.5, incl.	c						▒		▒	▒	▒
	—	120	0.25 to 1, incl.	c						▒		▒	▒	▒
A572 Gr. 42	42	60	to 6							▒		▒		▒
A572 Gr. 50	50	65	to 4							▒		▒		▒
A572 Gr. 55	55	70	to 2							▒		▒		▒
A572 Gr. 60	60	75	to 1.25							▒		▒		▒
A572 Gr. 65	65	80	to 1.25							▒		▒		▒
A588	42	63	Over 5 to 8, incl.							▒		▒		▒
	46	67	Over 4 to 5, incl.							▒		▒		▒
	50	70	4 and under							▒		▒		▒
A687	105	150 max.	0.625 to 3							▒		▒		▒
F1554 Gr. 36	36	58-80	0.25 to 4									■	■	■
F1554 Gr. 55	55	75-95	0.25 to 4									▒	▒	▒
F1554 Gr. 105	105	125-150	0.25 to 3									▒	▒	▒

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
☐ = Material specification does not apply.

— Indicates that a value is not specified in the material specification.
[a] Minimum unless a range is shown or maximum (max.) is indicated.
[b] Special washer requirements may apply per RCSC Specification Table 6.1 for some steel-to-steel bolting applications and per Part 14 for anchor-rod applications.
[c] See AISC Specification Section A3.3 for limitations on use of ASTM A449 bolts.
[d] When atmospheric corrosion resistance is desired, Type 3 can be specified.
[e] For anchor rods with temperature and corrosion resistance characteristics.

Table 7–20
Dimensions of Non-High-Strength Bolts, in.



Bolts Dia d_b, in.	Square			Hex			Heavy Hex			Countersunk		Min. Thrd. Length, in.	
	F, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*C*, in.	*H*, in.	$L \le$ 6 in.	$L >$ 6 in.
1/4	3/8	1/2	3/16	7/16	1/2	3/16	–	–	–	1/2	1/8	3/4	1
3/8	~~9/16~~	~~13/16~~	~~1/4~~	~~9/16~~	~~5/8~~	1/4	–	–	–	11/16	3/16	1	1 1/4
1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	1 1/8	1 1/8	5/16	1 1/2	1 3/4
3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
1	1 1/2	2 1/8	11/16	1 1/2	1 3/4	11/16	1 5/8	1 7/8	11/16	1 13/16	1/2	2 1/4	2 1/2
1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	11/16	3	3 1/4
1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
1 3/4	–	–	–	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	–	–	3 3/4	4
2	–	–	–	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	–	–	4 1/4	4 1/2
2 1/4	–	–	–	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	–	–	4 3/4	5
2 1/2	–	–	–	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	–	–	5 1/4	5 1/2
2 3/4	–	–	–	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	–	–	5 3/4	6
3	–	–	–	4 1/2	5 3/16	2	4 5/8	5 5/16	2	–	–	6	6 1/2
3 1/4	–	–	–	4 7/8	5 5/8	2 3/16	–	–	–	–	–	6	7
3 1/2	–	–	–	5 1/4	6 1/16	2 5/16	–	–	–	–	–	6	7 1/2
3 3/4	–	–	–	5 5/8	6 1/2	2 1/2	–	–	–	–	–	6	8
4	–	–	–	6	6 15/16	2 11/16	–	–	–	–	–	6	8 1/2

Notes:
For high-strength bolt and nut dimensions, refer to Table 7–15.
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.
Minimum thread length = $2d_b$ + 1/4-in. for bolts up to 6 in. long, and $2d_b$ + 1/2-in. for bolts longer than 6 in.

Eastern Engineering Group
3401 NW 82nd Avenue,
Suite 370, Miami, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

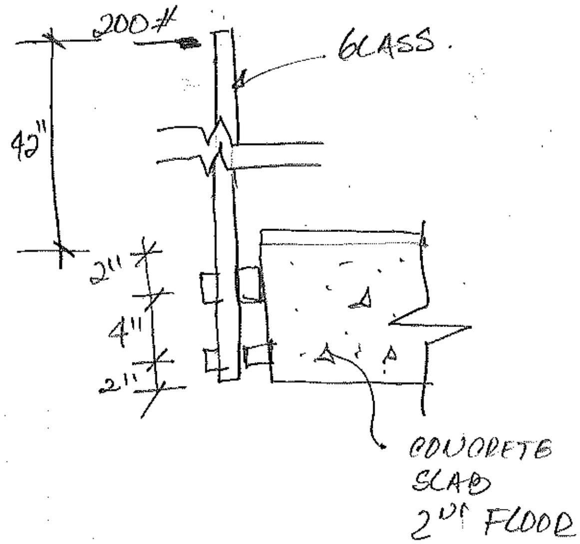
PROJ. NAME:

PROJ No.: DATE:

DESIGNED BY: PAGE:

REVIEWED BY:

CONNECTION TO CONCRETE SLAB



Load

P = 200#

Load processing

$T = \frac{P \times L}{d} = \frac{200\# \ (42"+2"+4)}{4}$

T = 2400#

* Asuming min (2) pair of stand-off per panel.

$T_{bolt} = \frac{2400\#}{2 \text{ bolt}} = 1200\#$ < 2034.77# =Tallow (OK)

$V \text{ per bolt} = \frac{200\#}{2} = 100\#$ < 524.16# = Vallow (OK)

(Conservative)

* Use ∅ 3/8" Threaded Rod C6+ Every Epoxy w/3-3/8" min embedment, 4" min spacing & 2" min edge dist.

Tbolt = 3080#x0.7379x0.8953
Tbolt = 2034.77#

Sbolt = 1300#x0.5166x0.7805
Sbolt = 524.16#

<u>COMBINED</u>

(Conserv.)

1200#/2034.77# + 200#/524.16#<1.00

0.97< 1.00 (OK)

PERFORMANCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod Ultimate Tension and Shear Loads[1,2,3] Installed in Solid Concrete

THREADED ROD DIA. (in.)	EMBEDMENT IN CONCRETE (in.)	MAX. CLAMPING FORCE AFTER PROPER CURE (ft./lbs.)	ULTIMATE TENSION (lbs.)			ULTIMATE SHEAR (lbs.)
			3,000 PSI CONCRETE	5,000 PSI CONCRETE	7,000 PSI CONCRETE	3,000 PSI CONCRETE & HIGHER
3/8	1-1/2	9	3,160	3,785	4,405	N/A
	3-3/8		11,640	12,315	12,985	5,200
1/2	2	16	6,075	7,015	7,950	N/A
	4-1/2		20,005	23,305	26,605	11,420
5/8	2-1/2	47	8,570	9,995	11,420	N/A
	5-5/8		24,905	29,015	33,125	18,300
3/4	3	70	12,030	13,570	15,105	N/A
	6-3/4		36,645	42,695	48,740	25,720
7/8	3-1/2	90	15,005	17,335	19,660	N/A
	7-7/8		55,575	70,338	85,100	32,120
1	4	110	17,735	20,390	23,045	N/A
	9		62,250	73,850	85,450	38,520
1-1/4	5	370	34,695	36,935	39,170	N/A
	11-1/4		77,815	90,655	103,495	65,080
1-1/2	13	450	101,085	117,765	134,445	N/A

1 Allowable working loads for the single installations under static loading should not exceed 25% capacity of the Ultimate Load. To calculate the Allowable Load of the anchor rod, divide the Ultimate Load by 4.
2 Performance values are based on the use of high strength threaded rod (ASTM A193 Gr. B7). The use of lower strength rods will result in lower ultimate tension and shear loads.
3 Linear interpolation may be used for intermediate spacing and edge distances.

PERFORMANCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod Allowable Tension Loads[1] Installed in Solid Concrete

THREADED ROD DIA in.	EMBEDMENT IN CONCRETE in.	ALLOWABLE TENSION LOAD BASED ON CONCRETE STRENGTH (lbs.)			ALLOWABLE TENSION LOAD BASED ON STEEL STRENGTH (lbs.)		
		3,000 psi concrete	5,000 psi concrete	7,000 psi concrete	ASTM A307	ASTM A193 GRADE B7	ASTM F593 AISI 304 SS
3/8	1-1/2	790	945	1,100	2,080	4,340	3,995
	3-3/8	2,910	3,080	3,245	2,080	4,340	3,995
1/2	2	1,520	1,755	1,990	3,730	7,780	7,155
	4-1/2	5,000	5,825	6,650	3,730	7,780	7,155
5/8	2-1/2	2,145	2,500	2,855	5,870	12,230	11,250
	5-5/8	6,225	7,255	8,280	5,870	12,230	11,250
3/4	3	3,010	3,395	3,775	8,490	17,690	14,860
	6-3/4	9,160	10,675	12,185	8,490	17,690	14,860
7/8	3-1/2	3,750	4,335	4,915	11,600	25,510	20,835
	7-7/8	13,895	17,585	21,275	11,600	25,510	20,835
1	4	4,435	5,100	5,760	15,180	31,620	26,560
	9	15,565	18,465	21,365	15,180	31,620	26,560
1-1/4	5	8,675	9,235	9,795	23,800	49,580	34,670
	11-1/4	19,455	22,665	25,875	23,800	49,580	34,670
1-1/2	13	25,270	29,440	33,610	33,720	70,250	47,770

1 Use lower value of either bond or steel strength for allowable tension load.

PERFORMANCE TABLE

C6+ For the Most Demanding Jobs

Threaded Rod Allowable Shear Loads[1] Installed in Solid Concrete

THREADED ROD DIA. (in.)	EMBEDMENT IN CONCRETE (in.)	ALLOWABLE SHEAR LOAD BASED ON CONCRETE STRENGTH (lbs.)	ALLOWABLE SHEAR LOAD BASED ON STEEL STRENGTH (lbs.)		
		3,000 psi concrete & higher	ASTM A307	ASTM A193 GRADE B7	ASTM F593 AISI 304 SS
3/8	1-1/2	N/A	1,040	2,170	1,995
	3-3/8	1,300	1,040	2,170	1,995
1/2	2	N/A	1,870	3,895	3,585
	4-1/2	2,855	1,870	3,895	3,585
5/8	2-1/2	N/A	2,940	6,125	5,635
	5-5/8	4,575	2,940	6,125	5,635
3/4	3	N/A	4,250	8,855	7,440
	6-3/4	6,430	4,250	8,855	7,440
7/8	3-1/2	N/A	5,800	12,760	10,730
	7-7/8	8,030	5,800	12,760	10,730
1	4	N/A	7,590	15,810	13,285
	9	9,630	7,590	15,810	13,285
1-1/4	5	N/A	11,900	24,790	18,840
	11-1/4	16,270	11,900	24,790	18,840

1 Use lower value of either concrete or steel strength for allowable shear load.

PERFORMANCE TABLE

C6+ For the Most Demanding Jobs

Rebar Ultimate Tension Loads[1,2,3] Installed in Solid Concrete

REINFORCING BAR	EMBEDMENT IN CONCRETE (in.)	ULTIMATE TENSION (lbs.)			ULTIMATE YIELD STRENGTH GRADE 60 REBAR (lbs.)	ULTIMATE TENSILE STRENGTH GRADE 60 REBAR (lbs.)
		3,000 psi concrete	5,000 psi concrete	7,000 psi concrete		
#3	1-1/2	3,160	3,785	4,405	6,600	9,900
	3-3/8	11,640	12,315	12,985		
#4	2	6,075	7,015	7,950	12,000	18,000
	4-1/2	20,005	23,305	26,605		
#5	2-1/2	8,570	9,995	11,420	18,600	27,900
	5-5/8	24,905	29,015	33,125		
#6	3	12,030	13,570	15,105	26,400	39,600
	6-3/4	36,645	42,695	48,740		
#7	3-1/2	15,005	17,335	19,660	36,000	54,000
	7-7/8	55,575	70,338	85,100		
#8	4	17,735	20,390	23,045	47,400	71,100
	9	62,250	73,850	85,450		
#10	5	34,695	36,935	39,170	79,200	114,300
	11-1/4	77,815	90,655	103,495		
#11	13	101,085	117,764	134,443	93,600	140,400

1 Allowable working loads for the single installation under static loading should not exceed 25% capacity of the Ultimate Load. To calculate the Allowable Load of the anchor, divide the ultimate load by 4.
2 Performance values are based on the use of ASTM A615 Grade 60 reinforcing bar. The use of lower strength rebar will result in lower ultimate tension loads
3 SHEAR DATA: Provided the distance from the rebar to the edge of the concrete member exceeds 1.25 times the embedment depth of the rebar, calculate the ultimate shear load for the rebar anchorage as 60% of the ultimate tensile strength of the rebar.

PERFORMANCE REFERENCE TABLE

C6+ For the Most Demanding Jobs

Threaded Rod and Rebar Installation in Solid Concrete Edge/Spacing Distance Load Factor Summary[1,2]

LOAD FACTOR	DISTANCE FROM EDGE OF CONCRETE	
Critical Edge Distance—Tension		
100% Tension Load →	1.25 x Anchor Embedment	4.21"
Minimum Edge Distance—Tension		
70% Tension Load →	0.50 x Anchor Embedment	1.68"
Critical Edge Distance—Shear		
100% Shear Load →	1.25 x Anchor Embedment	4.21"
Minimum Edge Distance—Shear		
30% Shear Load →	0.30 x Anchor Embedment	1.01"

LOAD FACTOR	DISTANCE FROM ANOTHER ANCHOR	
Critical Spacing—Tension		
100% Tension Load →	1.50 x Anchor Embedment (o	5.06"
Minimum Spacing—Tension		
75% Tension Load →	0.75 x Anchor Embedment	2.53"
Critical Spacing—Shear		
100% Shear Load →	1.50 x Anchor Embedment	5.06"
Minimum Spacing—Shear		
30% Shear Load →	0.50 x Anchor Embedment	1.68"

1 Use linear interpolation
2 Anchors are affected by ... the product of tension and sh

A	B
4.210	1
2.000	0.7379
1.680	0.7000

A	B
4.210	1
2.000	0.5166
1.010	0.3000

A	B
5.060	1
4.000	0.8953
2.530	0.7500

A	B
5.060	1
4.000	0.7805
1.680	0.3000

3401 NW 82nd Avenue
Ste. 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

EXTERIOR GLASS RAILING DESIGN

Glass Effective Laminated Thickness

Laminate Glass Data:

$E := 10400ksi$ — Young Modulus of Elasticity of Glass E MPa
For Soda Lime Glass: E=10 400 Ksi (71705.5 MPa)

$h_v := 0.06in$ — Interlayer Thickness (in)

$h_1 := 0.355 \cdot in$ — Glass Ply 1 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$h_2 := 0.355 \cdot in$ — Glass Ply 2 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$a := 30in$ — Length Scale (Smallest in-Plane Dimension of the Laminate Plate (in)

$G := 1640psi$ — Interlayer Storage Shear Modulus G=3830 psi (For Wind 3s/50 Celcius)
For Exterior: SentryGlass 1,640 psi (11.3 MPa) PVB 70 psi (0.48 MPa)
For Interior: SentryGlass 3830 psi (26.4 MPa) PVB 140 psi (0.96 MPa)

Then

$$h_s := 0.5 \cdot (h_1 + h_2) + h_v \qquad h_s = 0.415 \cdot in$$

$$h_{s.1} := \frac{h_s \cdot h_1}{h_1 + h_2} \qquad h_{s.1} = 0.207 \cdot in$$

$$h_{s.2} := \frac{h_s \cdot h_2}{h_1 + h_2} \qquad h_{s.2} = 0.207 \cdot in$$

$$I_s := h_1 \cdot h_{s.2}^{\ 2} + h_2 \cdot h_{s.2}^{\ 2} \qquad I_s = 0.031 \cdot in \cdot in \cdot in$$

$$\Gamma := \frac{1}{1 + \dfrac{9.6 \cdot E \cdot I_s \cdot h_v}{G \cdot h_s^{\ 2} \cdot a^2}} \qquad \Gamma = 0.581$$

Laminate Effective Thickness For Deflection

$$h_{efw} := \sqrt[3]{h_1^{\ 3} + h_2^{\ 3} + 12 \cdot \Gamma \cdot I_s} \qquad h_{efw} = 0.671 \cdot in$$

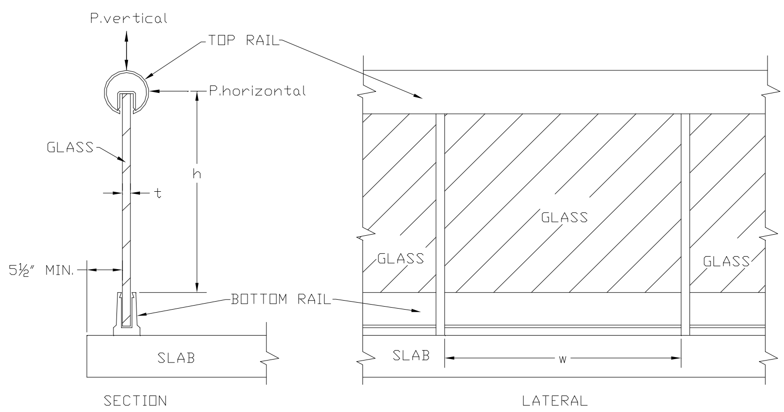
Laminate Effective Thickness For Bending Stress

$$h_{1.ef\sigma} := \sqrt{\frac{h_{efw}^{\ 3}}{\max(h_1, h_2) + 2 \cdot \Gamma \cdot \max(h_{s.1}, h_{s.2})}} \qquad h_{1.ef\sigma} = 0.713 \cdot in$$

TABLE 4 Nominal and Minimum Glass Thicknesses

Nominal Thickness or Designation, mm (in.)	Minimum Thickness, mm (in.)
2.5 (3⁄32)	2.16 (0.085)
2.7 (lami)	2.59 (0.102)
3.0 (1⁄8)	2.92 (0.115)
4.0 (5⁄32)	3.78 (0.149)
5.0 (3⁄16)	4.57 (0.180)
6.0 (1⁄4)	5.56 (0.219)
8.0 (5⁄16)	7.42 (0.292)
10.0 (3⁄8)	9.02 (0.355)
12.0 (1⁄2)	11.91 (0.469)
16.0 (5⁄8)	15.09 (0.595)
19.0 (3⁄4)	18.26 (0.719)
22.0 (7⁄8)	21.44 (0.844)
25.0 (1)	24.61 (0.969)

Glass Railing Design w/ Base Shoe Support 2025



Loads Data:

$P_{200} := 200.00$ Concentrated Load (lbs)

$q_{50} := 50.00$ Uniform Load (plf)

$q_{wind} := 54.24$ Uniform Distributed Load (psf)

Glass Data:

$E := 10400000.00$ Modulus of Elasticity of Glass (psi)

$Mr_{flexure} := 24000.00$ Modulus of Rupture of Glass in Flexure For Live Loads (psi)

$F_{b.wind} := 9600.00$ Allowable Stress in Flexure For Wind Loads Per ASTM E1300 (psi)

$Mr_{shear} := 12000.00$ Modulus of Rigity of glass in Shear (psi)

$SF := 4.00$ Safety Factor

Geometric Glass Railing Data:

$h := 37.25$ Height of Glass Lite Pannel in Cantilever (in) (Conserv.)

$t_s := 0.713$ Effective Thickness of Glass Pannel for Stress (in) Per ASTM E1300 Appendix X9

$t_d := 0.671$ Effective Thickness of Glass Pannel for Deflection (in) Per ASTM E1300 Appendix X9

$w_{min} := 30.00$ **Minimum** Width of Glass Pannel (in)

$w_{max} := 60.00$ **Maximum** Width of Glass Pannel (in)

Then

$$F_{b.live} := \frac{Mr_{flexure}}{SF}$$ $F_{b.live} = 6000.00$ psi

$$F_v := \frac{Mr_{shear}}{SF}$$ $F_v = 3000.00$ psi

$$L := \begin{cases} h \\ w_{min} & \text{if } w_{min} < h \end{cases}$$ $L = 30.00$ in

$$S_x := \frac{L \cdot t_s^2}{6}$$ $S_x = 2.54$ in^3

$$I_x := \frac{L \cdot t_s^3}{12}$$ $I_x = 0.91$ in^4

$$A := L \cdot t_s$$ $A = 21.39$ in^2

Actual Glass Moment in Length (L):

Concentrated Load = 200 lbs.

$$M_{200} := P_{200} \cdot h$$ $M_{200} = 7450.00$ lbs – in

Uniform Load = 50 plf

$$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$$ $M_{50} = 4656.25$ lbs – in

Uniform Distributed Wind Load

$$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$$ $M_{wind} = 7839.73$ lbs – in

$$M_{Glass.live} := \max(M_{200}, M_{50})$$ $M_{Glass.live} = 7450.00$ lbs – in

$$M_{Glass.wind} := M_{wind}$$ $M_{Glass.wind} = 7839.73$ lbs – in

$$M_{Glass} := \max(M_{Glass.live}, M_{Glass.wind})$$ $M_{Glass} = 7839.73$ lbs – in

Actual Glass Shear in Length (L):

Concentrated Load = 200 lbs.

$$V_{200} := P_{200}$$ $V_{200} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50} := q_{50} \cdot \frac{L}{12}$$ $V_{50} = 125.00$ lbs

Uniform Distributed Wind Load

$$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$$ $V_{wind} = 420.93$ lbs

$$V_{Glass} := \max\left(V_{200}, V_{50}, V_{wind}\right)$$ $V_{Glass} = 420.93$ lbs

Section Required:

Bending Design:

Section Modulus Required

$$Sx_r := \max\left(\frac{M_{Glass.live}}{F_{b.live}}, \frac{M_{Glass.wind}}{F_{b.wind}}\right)$$ $Sx_r = 1.24$ in^3

Shear Design:

Area Required

$$A_r := \frac{V_{Glass}}{F_v}$$ $A_r = 0.14$ in^2

Section Provided:

$$BENDING_{glass} := \text{if}\left(Sx_r \geq \min\left(S_x\right), \text{"N.G"}, \text{"OK"}\right)$$ $BENDING_{glass} = \text{"OK"}$

$$SHEAR_{glass} := \text{if}\left(A_r \geq A, \text{"N.G"}, \text{"OK"}\right)$$ $SHEAR_{glass} = \text{"OK"}$

Check Molding Walls on Base:

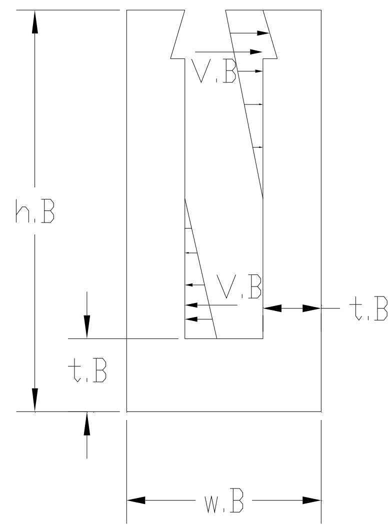
$h_B := 4.75$ Height of Bottom Rail (in)

$t_B := 0.75$ Thick of Wall in Bottom Rail (in)

$w_B := 3.25$ Width of Bottom Rail (in)

$F_{b.} := 9500.00$ Allowable Bending Stress in Bottom Rail (psi)

$F_{v.} := 5500.00$ Allowable Shear Stress in Bottom Rail (psi)



Alloy 6063-T5

Actual Shear on Wall`s Molding Per Length (L):

Concentrated Load = 200 lbs.

$$V_{W200} := \frac{M_{200}}{(h_B - t_B)} + P_{200}$$

$V_{W200} = 2062.50$ lbs

Uniform Load = 50 plf

$$V_{W50} := \frac{M_{50}}{(h_B - t_B)} + q_{50} \cdot \frac{L}{12}$$

$V_{W50} = 1289.06$ lbs

Uniform Distributed Wind Load

$$V_{Wwind} := \frac{M_{wind}}{(h_B - t_B)} + \frac{q_{wind}}{144} \cdot L \cdot (h + h_B)$$

$V_{Wwind} = 2434.53$ lbs

$$V_{Wall} := \max(V_{W200}, V_{W50}, V_{Wwind})$$

$V_{Wall} = 2434.53$ lbs

Geometric Inertia & Area in Wall`s Molding:

$$S_B := \frac{L \cdot t_B^{\ 2}}{6}$$ $S_B = 2.81$ in^3

$$A_B := L \cdot t_B$$ $A_B = 22.50$ in^2

Check Bending Stress in Molding Wall:

$$f_b := \frac{M_{Glass}}{S_B}$$ $f_b = 2787.46$ psi

$$BENDING := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } f_b \le F_{b.} \end{array} \right.$$ BENDING = "OK"

Check Shear Stress in Molding Wall:

$$f_v := \frac{V_{Wall}}{A_B}$$ $f_v = 108.20$ psi

$$SHEAR := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } f_v \le F_{v.} \end{array} \right.$$ SHEAR = "OK"

Check Combined Bending/Shear in Wall`s Monting:

$$C_{BV} := \frac{f_b}{F_{b.}} + \frac{f_v}{F_{v.}}$$ $C_{BV} = 0.31$

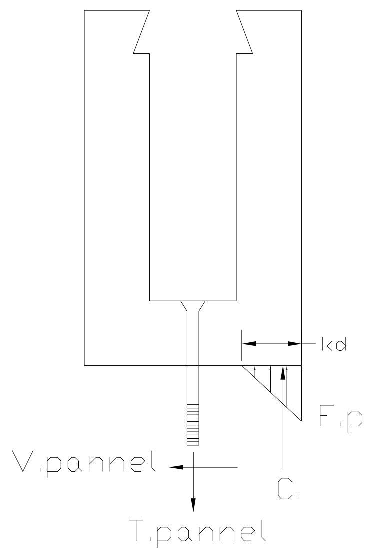
$$COMBINED := \left| \begin{array}{l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } C_{BV} \le 1.00 \end{array} \right.$$ COMBINED = "OK"

Base Shoe Connection Design:

Check Anchors:

$s_{bolt} := 6.0$ Spacing of Bolts Per Pannels

$F_p := 2100.00$ Allowable Compressive Strength in Support (psi)



$$L_1 := \begin{cases} h + h_B \\ w_{max} \text{ if } w_{max} < h + h_B \end{cases}$$

$L_1 = 42.00$ in

Actual Base Moment in Length (L1):

Concentrated Load = 200 lbs.

$$M_{B200} := P_{200} \cdot (h + h_B)$$

$M_{B200} = 8400.00$ lbs − in

Uniform Load = 50 plf

$$M_{B50} := q_{50} \cdot \frac{L_1}{12} \cdot (h + h_B)$$

$M_{B50} = 7350.00$ lbs − in

Uniform Distributed Wind Load

$$M_{Bwind} := \frac{q_{wind}}{144} \cdot L_1 \cdot \frac{(h + h_B)^2}{2}$$

$M_{Bwind} = 13953.24$ lbs − in

$$M_{Base} := \max(M_{B200}, M_{B50}, M_{Bwind})$$

$M_{Base} = 13953.24$ lbs − in

Actual Base Shear in Length (L1):

Concentrated Load = 200 lbs.

$$V_{B.200} := P_{200}$$ $V_{200} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{B.50} := q_{50} \cdot \frac{L_1}{12}$$ $V_{50} = 125.00$ lbs

Uniform Distributed Wind Load

$$V_{B.wind} := \frac{q_{wind}}{144} \cdot L_1 \cdot (h + h_B)$$ $V_{B.wind} = 664.44$ lbs

$$V_{Base} := \max(V_{B.200}, V_{B.50}, V_{B.wind})$$ $V_{Base} = 664.44$ lbs

Length of Compression Zone:

$$kd := \left[\frac{F_p \cdot L_1 \cdot (0.5 w_B)}{2} - \sqrt{\frac{(F_p \cdot L_1)^2 \cdot (0.5 w_B)^2}{4} - \frac{4 \cdot (M_{Base}) \cdot F_p \cdot L_1}{6}} \right] \cdot \frac{3}{F_p \cdot L_1}$$ $kd = 0.20$ in

Tensile Load on Cap Screws Per Width of Glass Pannel:

$$T_{pannel} := 0.5 \cdot F_p \cdot kd \cdot L_1$$ $T_{pannel} = 8960.04$ lbs

Shear Load on Cap Screws Per Width of Glass Pannel:

$$V_{pannel} := V_{Base}$$ $V_{pannel} = 664.44$ lbs

Check Compressive Stress in Support:

$$f_p := \text{floor}\left(\frac{2 T_{pannel}}{kd \cdot L_1} \right)$$ $f_p = 2100.00$ psi

$$\text{COMPRESSION} := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_p \geq f_p \end{vmatrix}$$ COMPRESSION = "OK"

Number of Anchors Per Loaded Triangle:

$$N_{anchors} := \mathrm{floor}\left(\frac{L_1}{s_{bolt}}\right) + 1$$

$\boxed{N_{anchors} = 8.00}$ bolts

Check Tension in Anchors:

$$T_d := \frac{T_{pannel}}{N_{anchors}}$$

$\boxed{T_d = 1120.00}$ lbs

Check Shear in Anchors:

$$V_d := \frac{V_{pannel}}{L_1} \cdot s_{bolt}$$

$\boxed{V_d = 94.92}$ lbs

Tbolt = (1120# x 3 bolt) ÷ 0.6 = 5600# (LRFD)
Sbolt = (94.92# x 3 bolt) ÷ 0.6 = 475# (LRFD)

(See next Truspec analysis)

Use Ø3/8" threaded rod A193 B7 w/c6+ epoxy adhesive by red head w/5-13/16" min. embedment, 6" spacing & 4" min. edge distance.

CALCULATION SHEET FOR RED HEAD ANCHORS

Company :

Carried out by :

Phone number :

Mail address :

Company :	Eastern Engineering Group	Project name :	26-0297
Contact name :	Maria Peroza	Location :	
Phone number :	3055998133	Fastening point :	
Mail address :	maria.e@easterneg.com		
Comment :			

Recommended anchors

C6+ Threaded Rod A193 B7 Carbon Steel 3/8 x 5-49/59

Min. Effective embedment: 5.831 in

ESR 4046 issued 2025/09/01 / 2026/09/01

Base material

Concrete Strength	5000 psi
Concrete condition:	Cracked
Thickness of concrete :	12 in
Tension load conditions:	Condition A Tension
Shear load conditions:	Condition A Shear
Edge reinforcement :	None or < no. 4 bar

Longitudinal reinforcement should be provided along the edge of the member

Conditions

Installation conditions :	Dry concrete
Hole drilling method:	Hammer drilling
Short term temperature :	130 °F
Long term temperature :	110 °F

Anchor plate

Thickness of part to be fixed :	0.7 in
Recommended plate thickness :	The base plate thickness has not been checked
Clearance diameter :	0.5 in
Profile :	
Profile position :	Ex = 0 in ; Ey = 0 in
Stand-off :	None

Design method : ACI 318-19 Design for static, quasi-static loading

Design Actions :

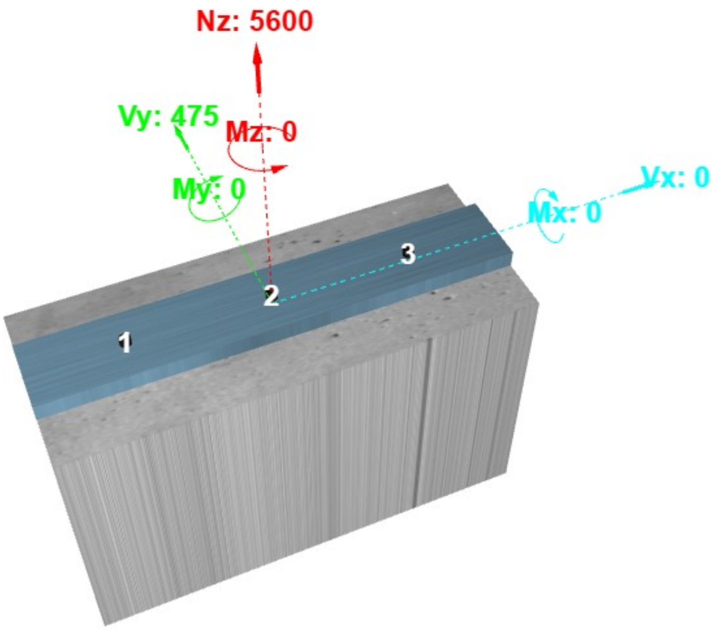
Action [lbf] / [lbf ft]	Action type	Nuz	Vux	Vuy	Muz	Mux	Muy
Combination 1	Static	5600	0	475	0	0	0

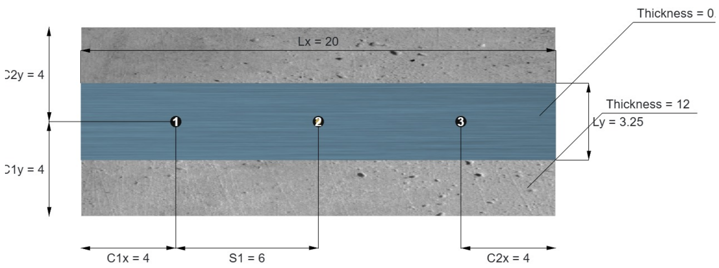
Specifications :

Static

Sustained Load : False

Geometry :





Calculation Hypothesis :

- The anchoring plate is assumed to be sufficient to resist deformation imposed by the load actions.

- REDHEAD can only be held responsible if the calculation examples exactly reflect the application and if the installation is carried out according to the instruction given in the REDHEAD specifications. The calculation is correct for REDHEAD anchors only. The
contractor or specifier should make sure that the base material is able to support the loads especially in the case of a group of anchors. REDHEAD cannot be held responsible if this software package is modified without its written approval.

truspec ANCHOR DESIGN SOFTWARE

Resulting anchors forces

Loads on anchors

Anchor	Tensile	Shear[x]	Shear[y]
1	1867 lbf	0 lbf	158 lbf
2	1867 lbf	0 lbf	158 lbf
3	1867 lbf	0 lbf	158 lbf

N^g [lbf]	N^h [lbf]	e_{Nx}[in]	e_{NY}[in]
5600	1867	0	0

V^h [lbf]	V^h [lbf]
475	158

Utilization

Tension load	Tension force N_{ua} [lbf]	Capacity ΦN_n [lbf]	$\beta_N = N_{ua} / \Phi N_n$ [%]
Bond strength	5600	5839	95.9
Sustained Tension Load Bond Strength	/	/	/
Concrete breakout strength	5600	9853	56.8
Steel strength	1867	7267	25.7

Shear load	Shear force V_{ua} [lbf]	Capacity ΦV_n [lbf]	$\beta_V = V_{ua} / \Phi V_n$ [%]
Concrete Breakout Strength	475	4135	11.49
Pryout strength	475	13474	3.53
Steel strength	158	3776	4.19

Combined tension and shear loads

$$\beta_{Nc}^{5/3} + \beta_{Vc}^{5/3} = [0.96]^{5/3} + [0.11]^{5/3} = 0.96 \leq 1$$

CALCULATION DETAILS

Tension load - Bond Strength

$\Phi N_{ag} \geq N_{ua}$	[ACI 318−19 Table.17.5.2]
$N_{ag} = N_{ba} \cdot \frac{A_{Na}}{A_{a0}} \cdot \Psi_{ec,Na} \cdot \Psi_{ed,Na} \cdot \Psi_{cp,Na}$	[ACI 318−19 Eq. (17.6.5.1b)]
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \Pi \cdot d_a \cdot h_{ef}$	[ACI 318−19 Eq. (17.6.5.2.1)]
A_{Na}	[ACI 318−19 Section 17.6.2.1 – Fig. R17.6.2.1(b)]
$A_{Na0} = (2 \cdot c_{Na})^2$	[ACI 318−19 Eq. (17.6.5.1.2a)]
$c_{Na} = 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100}}$	[ACI 318−19 Eq. (17.6.5.1.2a)]
$\Psi_{ed,Na} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{c_{Na}} \leq 1$	[ACI 318−19 Eq. (17.6.5.4.1a)]
$\Psi_{cp,Na} = \max\left(\frac{c_{a,min}}{c_{ac}}; \frac{c_{Na}}{c_{ac}}\right) \leq 1$	[ACI 318−19 Eq. (17.4.5.5.1b)]
$\Psi_{ec,Na} = \frac{1}{1 + (2 \cdot e_N / c_{Na})} \leq 1$	[ACI 318−19 Eq. (17.6.5.3.1)]

ΦN_{ag}	= 5839 lbf	N_{ba}	= 7728 lbf	f'_c	= 5000 psi
N_{ag}	= 8983 lbf	$\tau_{k,c}$	= 1125 psi	λ_a	= 1
Φ_{bond}	= 0.65	τ_{uncr}	= 2470 psi	c_{Na}	= 5.6 in
		A_{Na}/A_{Na0}	= 1.27	d_a	= 0.375 in
		A_{Na}	= 161 in²	h_{ef}	= 5.831 in
		A_{Na0}	= 126 in²	c_{ac}	= 13.1 in
		$\Psi_{ecx,Na}$	= 1.00	e_{Nx}	= 0 in
		$\Psi_{ecy,Na}$	= 1.00	e_{Ny}	= 0 in
		$\Psi_{ed,Na}$	= 0.91		
		$\Psi_{cp,Na}$	= 1.00		

Tension load - Concrete breakout Strength

$\Phi N_{cbg} \geq N_{ua}$	[ACI 318−19 Table.17.5.2]
$N_{cbg} = N_b \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N}$	[ACI 318−19 Eq.(17.6.2.1b)]
$N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$	[ACI 318−19 Eq.(17.6.2.2.1)]
A_{Nc}	[ACI 318−19 Section 17.6.2.1 − Fig. R17.6.2.1(b)]
$A_{Nc0} = 9 \cdot h_{ef}^2$	[ACI 318−19 Section 17.6.2.1 −Fig. R17.6.2.1(a)]
$\Psi_{ed,N} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1$	[ACI 318−19 Eq.(17.6.5.4.1)]
$\Psi_{cp,N} = \max\left(\frac{c_{a,min}}{c_{ac}}; \frac{1.5 \cdot h_{ef}}{c_{ac}}\right) \leq 1$	[ACI 318−19 Eq.(17.6.5.5.1]
$\Psi_{ec,N} = \frac{1}{1 + (2 \cdot e_N / 3 \cdot h_{ef})} \leq 1$	[ACI 318−19 Eq.(17.6.5.3.1)]

ΦN_{cbg}	= 9853 lbf	N_b	= 5234 lbf	k_c	= 17
N_{cbg}	= 13137 lbf	A_{Nc}/A_{Nc0}	= 2.51	f'_c	= 5000 psi
$\Phi_{concrete}$	= 0.75	A_{Nc}	= 161 in²	λ_a	= 1
		A_{Nc0}	= 64 in²	$\Psi_{c,N}$	= 1.00
		$\Psi_{ec,Nx}$	= 1.00	h_{ef}	= 5.831 in
		$\Psi_{ec,Ny}$	= 1.00	c_{ac}	= 6 in
		$\Psi_{ed,N}$	= 1.00	e_{Nx}	= 0 in
		$\Psi_{cp,N}$	= 1.00	e_{Ny}	= 0 in

Tension load - Steel Strength

$\Phi N_{sa} \geq N_{ua}$	[ACI 318−19 Table.17.5.2]
N_{sa}	[ESR approval]

ΦN_{sa}	= 7267 lbf
N_{sa}	= 9690 lbf
Φ_{steel}	= 0.75

Shear load - Concrete Breakout Strength

$\Phi V_{cbg} \geq V_{ua}$	[ACI 318−19 Table.17.5.2]
$V_{cbg} = V_b \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ed,V} \cdot \Psi_{h,V} \cdot \Psi_{ec,V} \cdot \Psi_{c,V} \cdot \Psi_{parallel,V}$	[ACI 318−19 – Eq. (17.7.2.1b)]
$V_b = \min\left[\left(7 \cdot \left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{d_a}\right) \cdot \lambda_a \cdot \sqrt{f_c} \cdot c_{a1}^{1.5}; 9 \cdot \lambda_a \cdot \sqrt{f_c} \cdot c_{a1}^{1.5}\right]$	[ACI 318−19 – Eq. (17.7.2.2.1a)]
A_{Vc}	[ACI 318−19 Section 17.7.2.1.1 – Fig. R17.7.2.1(b)]
$A_{Vc0} = 4.5 \cdot c_{a1}^2$	[ACI 318−19 – Eq. (17.7.2.1.3)]
$\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}} \leq 1$	[ACI 318−19 – Eq. (17.7.2.4.1a)]
$\Psi_{h,V} = \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}} \geq 1$	[ACI 318−19 – Eq. (17.7.2.6.1)]
$\Psi_{ec,V} = \frac{1}{1 + 2 \cdot (e_V / 3 \cdot c_{a1})} \leq 1$	[ACI 318−19 – Eq. (17.7.2.3.1)]

ΦV_{cbg}	= 4135 lbf	V_b	= 3675 lbf	c_{a1}	= 4 in
V_{cbg}	= 5513 lbf	A_{Vc}/A_{Vc0}	= 1.67	c_{a2}	= 4 in
$\Phi_{concrete}$	= 0.75	A_{Vc}	= 120 in²	f'_c	= 5000 psi
		A_{Vc0}	= 72 in²	λ_a	= 1
		$\Psi_{ec,V}$	= 1.00	l_e	= 3 in
		$\Psi_{ed,V}$	= 0.90	d_a	= 0.375 in
		$\Psi_{h,V}$	= 1.00	$e_{c,V}$	= 0 in
		$\Psi_{c,V}$	= 1.00		
		$\Psi_{parallel,V}$	= 1.00		

Shear load - Pryout Strength based on Concrete Strength

$\Phi V_{cpg} \geq V_{ua}$	[ACI 318−19 Table.17.5.2]
$V_{cpg} = k_{cp} \cdot \left(N_{ba} \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ec,Na} \cdot \Psi_{ed,Na} \cdot \Psi_{cp,N} \right)$	[ACI 318−19 – Eq. (17.7.3.1b)]
$N_b = \lambda_a \cdot \tau_{k,c} \cdot \Pi \cdot d_a \cdot h_{ef}$	[ACI 318−19 – Eq. (17.6.5.2.1)]
A_{Na}	[ACI 318−19 Section 17.6.5.1.2 – Fig. R17.6.5.1(b)]
$A_{Na0} = (2.c_{Na})^2$	[ACI 318−19 – Eq. (17.6.2.1a)]
$\Psi_{ed,Na} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{c_{Na}} \leq 1$	[ACI 318−19 – Eq. (17.6.2.4.1)]
$\Psi_{cp,Na} = \max\left(\frac{c_{a,min}}{c_{ac}}; \frac{c_{Na}}{c_{ac}} \right) \leq 1$	[ACI 318−19 – Eq. (17.6.2.6.1]
$\Psi_{ec,Na} = \frac{1}{1 + (2 \cdot e_N / c_{Na})} \leq 1$	[ACI 318−19 – Eq. (17.6.2.3.1)]

ΦV_{cpg}	= 13474 lbf	N_{ba}	= 7728 lbf	f'_c	= 5000 psi
V_{cpg}	= 17966 lbf	$\tau_{k,c}$	= 1125 psi	λ_a	= 1
$\Phi_{concrete}$	= 0.75	τ_{uncr}	= 2470 psi	c_{Na}	= 5.6 in
		A_{Na}/A_{Na0}	= 1.27	d_a	= 0.375 in
		A_{Na}	= 161 in²	h_{ef}	= 5.831 in
		A_{Na0}	= 126 in²	c_{ac}	= 6 in
		$\Psi_{ecx,Na}$	= 1.00	e_{Nx}	= 0 in
		$\Psi_{ecy,Na}$	= 1.00	e_{Ny}	= 0 in
		$\Psi_{ed,Na}$	= 0.91		
		$\Psi_{cp,Na}$	= 1.00		

Shear load - Steel Strength

$\Phi V_{sa} \geq V_{ua}$	[ACI 318−19 Table.17.5.2]
V_{sa}	[ESR approval]

ΦV_{sa}	= 3776 lbf
V_{sa}	= 5810 lbf
Φ_{steel}	= 0.65

INSTALLATION DATA

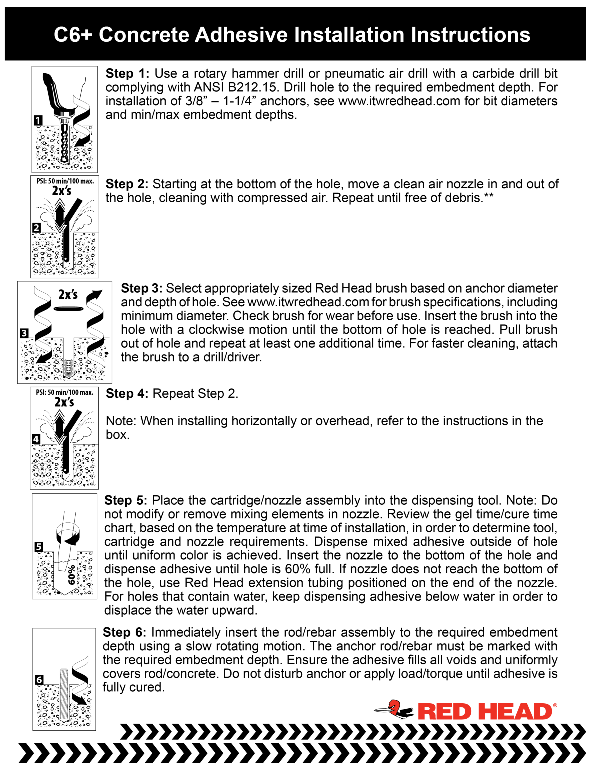
C6+ Threaded Rod A193 B7 Carbon Steel 3/8 x 5-49/59

Min. Effective embedment: **5.831 in**

ESR 4046 issued 2025/09/01 / 2026/09/01

Min. Effective embedment:	5.831 in
Minimum thickness of base material :	7 in
Hole diameter in the base material :	0.4 in
Min. hole depth in base material	5.8307 in
Design installation torque	15.00 ft-lb
Max. base plate thickness	0.7 in
Profile family (section type) :	
Clearance diameter :	0.5 in

INSTALLATION Method



C6+ Concrete Adhesive Installation Instructions

Step 1: Use a rotary hammer drill or pneumatic air drill with a carbide drill bit complying with ANSI B212.15. Drill hole to the required embedment depth. For installation of 3/8" – 1-1/4" anchors, see www.itwredhead.com for bit diameters and min/max embedment depths.

Step 2: Starting at the bottom of the hole, move a clean air nozzle in and out of the hole, cleaning with compressed air. Repeat until free of debris.**

Step 3: Select appropriately sized Red Head brush based on anchor diameter and depth of hole. See www.itwredhead.com for brush specifications, including minimum diameter. Check brush for wear before use. Insert the brush into the hole with a clockwise motion until the bottom of hole is reached. Pull brush out of hole and repeat at least one additional time. For faster cleaning, attach the brush to a drill/driver.

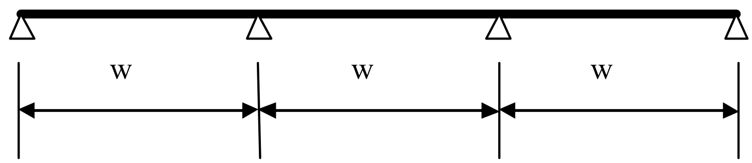
Step 4: Repeat Step 2.

Note: When installing horizontally or overhead, refer to the instructions in the box.

Step 5: Place the cartridge/nozzle assembly into the dispensing tool. Note: Do not modify or remove mixing elements in nozzle. Review the gel time/cure time chart, based on the temperature at time of installation, in order to determine tool, cartridge and nozzle requirements. Dispense mixed adhesive outside of hole until uniform color is achieved. Insert the nozzle to the bottom of the hole and dispense adhesive until hole is 60% full. If nozzle does not reach the bottom of the hole, use Red Head extension tubing positioned on the end of the nozzle. For holes that contain water, keep dispensing adhesive below water in order to displace the water upward.

Step 6: Immediately insert the rod/rebar assembly to the required embedment depth using a slow rotating motion. The anchor rod/rebar must be marked with the required embedment depth. Ensure the adhesive fills all voids and uniformly covers rod/concrete. Do not disturb anchor or apply load/torque until adhesive is fully cured.

Top Railing Design:



Handrail to provide redistribution of load between glass panels & to remain in place in case that one of the glass planes breaks

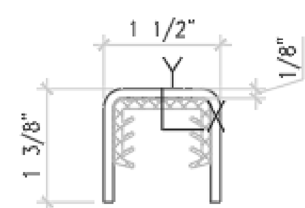
$F_{b.top} := 34800.00$ Top Railing Allowable Bending Stress (psi)

$F_{v.top} := 20830.00$ Top Railing Allowable Shear Stress (psi)

$Sx_{top} := 0.1225$ Top Railing Inertia Modulus for Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.3164$ Top Railing Inertia Modulus for Horizontal Loads $\left(in^3\right)$

$A_{top} := 0.7799$ Top Railing Area $\left(in^2\right)$



Area = 0.7799 in^2
Ixx = 0.1108 in^4
Iyy = 0.2303 in^4
Jz = 0.3412 in^4
Pxy = −0.0000 in^4
rxx = 0.3770 in
ryy = 0.5434 in
Sxx max = 0.2168 in^3
Sxx min = 0.1225 in^3
Syy max = 0.3164 in^3
Syy min = 0.3164 in^3

Maximum Moment:

Concentrated Load = 200 lbs.

$$M_{200.top} := \frac{P_{200} \cdot w_{max}}{5}$$ $M_{200.top} = 2400.00$ lb − in

Uniform Load = 50 plf

$$M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot w_{max}^{\ 2}$$ $M_{50.top} = 1518.00$ lb − in

$$M_{actual.top} := \max\left(M_{200.top}, M_{50.top}\right)$$ $M_{actual.top} = 2400.00$ lb − in

Maximum Shear:

Concentrated Load = 200 lbs.

$$V_{200.top} := P_{200}$$ $V_{200.top} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot w_{max}$$ $V_{50.top} = 150.00$ lbs

$$V_{actual.top} := \max(V_{200.top}, V_{50.top})$$ $V_{actual.top} = 200.00$ lbs

Section Required:

Bending Design:
Section Modulus Required

$$Shr := \frac{M_{actual.top}}{F_{b.top}}$$ $Shr = 0.07$ in^3

Shear Design:
Area Required

$$Ahr := \frac{1.5 \cdot V_{actual.top}}{F_{v.top}}$$ $Ahr = 0.01$ in^2

Section Provided:

$$BENDING_{top} := if(Shr \geq \min(Sx_{top}, Sy_{top}), "N.G", "OK")$$ $BENDING_{top} = "OK"$

$$SHEAR_{top} := if(Ahr \geq A_{top}, "N.G", "OK")$$ $SHEAR_{top} = "OK"$