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Engineering Group

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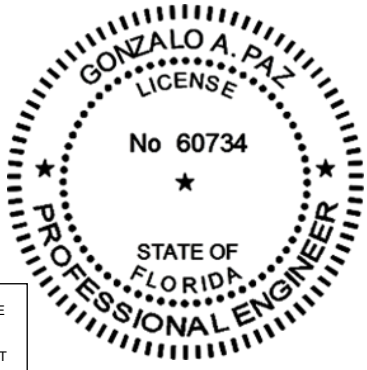
6200 SW 123RD TERRACE

Job. No: 25-0925

MISC. SHOP DRAWINGS

6200 SW 123rd Terrace,
Miami, FL 33156, USA

STRUCTURAL CALCULATION



Digitally signed by Gonzalo A Paz
Date: 2025.09.29 11:27:54-04'00'

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Prepared By:
☐ Gonzalo Paz , PE
Lic. No. 60734
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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. Specifications for the Design of Cold-Formed Stainless Steel Structural Members SEI/ASCE8-02

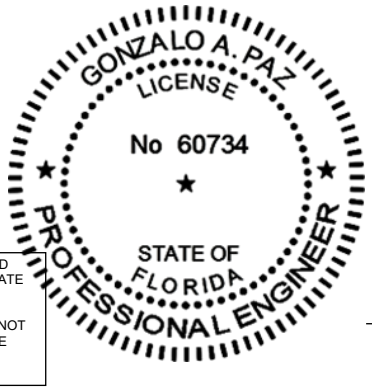
CALCULATION INDEX:

I.	Wind Load Analysis	3-4
II.	Central Stringer Stair Design	5-28
III.	Stair Connections Design	29-77
IV.	Glass Railing At Stair Design	78-96
V.	Handrail At Wall Design	97-106
VI.	Cable Railings At Balconies Design	107-131

Total Pages = 131

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.



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WIND LOADS ANALYSIS

WIND ANALYSIS FOR CABLE RAILINGS

Wind for Others Structures & Opening Signs Design (ε<70%) ASCE 7-22

General Wind Data:

Miami Dade

$V := 175.00$ Wind Velocity (mph) **Category II**.....V=175 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 (Rigid Structure)

$C_f := 1.80$ Net Force Coefficients Cf (see Figure 29.5-2)

	Flat-Side Members	Rounded Members
for ε<=0.1----------	2.0	1.2----------0.8
for 0.1 to 0.29----------	1.8	1.3----------0.9
for 0.3 to 0.7----------	1.6	1.5----------1.1

$\alpha := 9.80$

$zg := 2460.00$

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:

$Z := 16.33$ Height of Top of Sign (ft)

$\varepsilon := 10.00$ Solidity Ratio of Sign (%)

Then

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

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

$K_z = 0.87$

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

$q_z = 57.71$ psf

Allowable Design Wind Loads:

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

$p_z = 52.98$ Gross Wind per Actual Solid Area (psf)

$p_{z.EQ} := \max\left(p_z \cdot \frac{\varepsilon}{100}, 10\right)$

$p_{z.EQ} = 10.00$ Equivalent Wind in Overall Area (psf)

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CENTRAL STRINGER STAIR DESIGN

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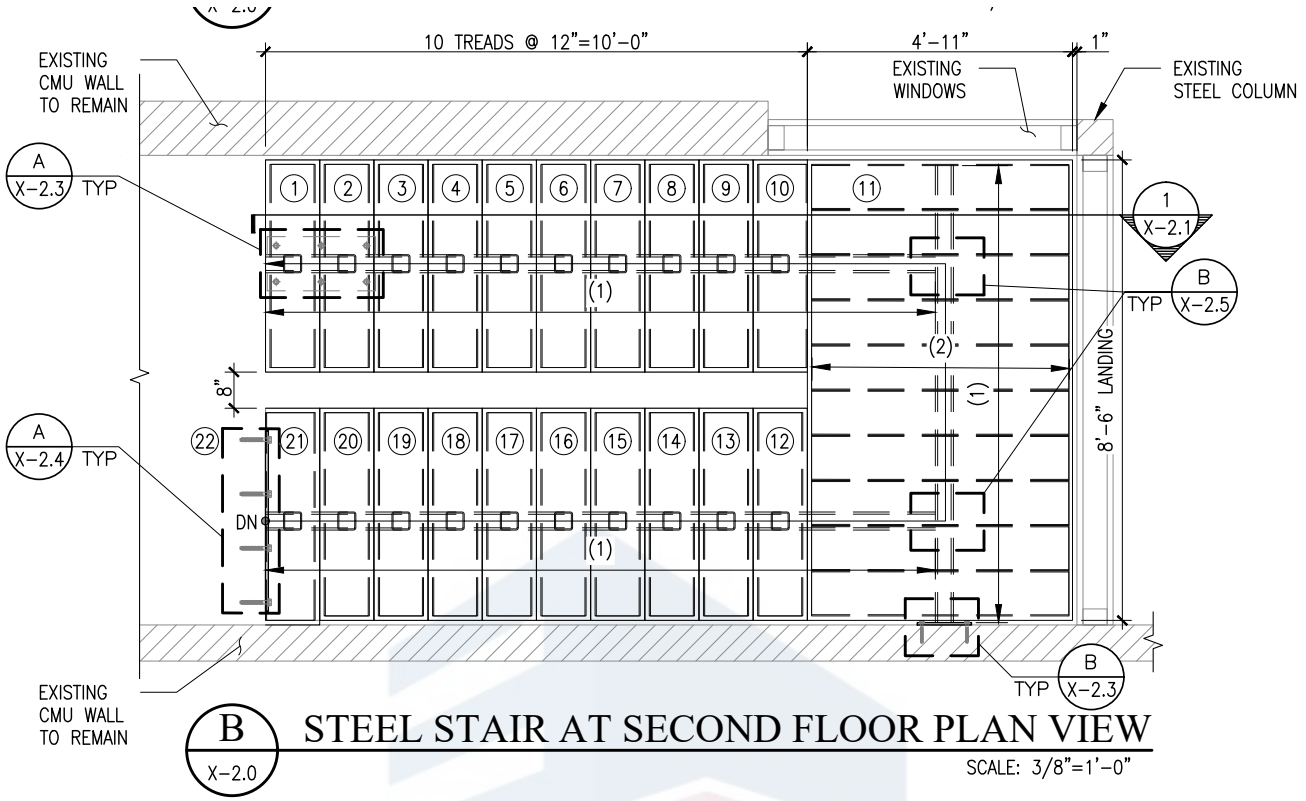
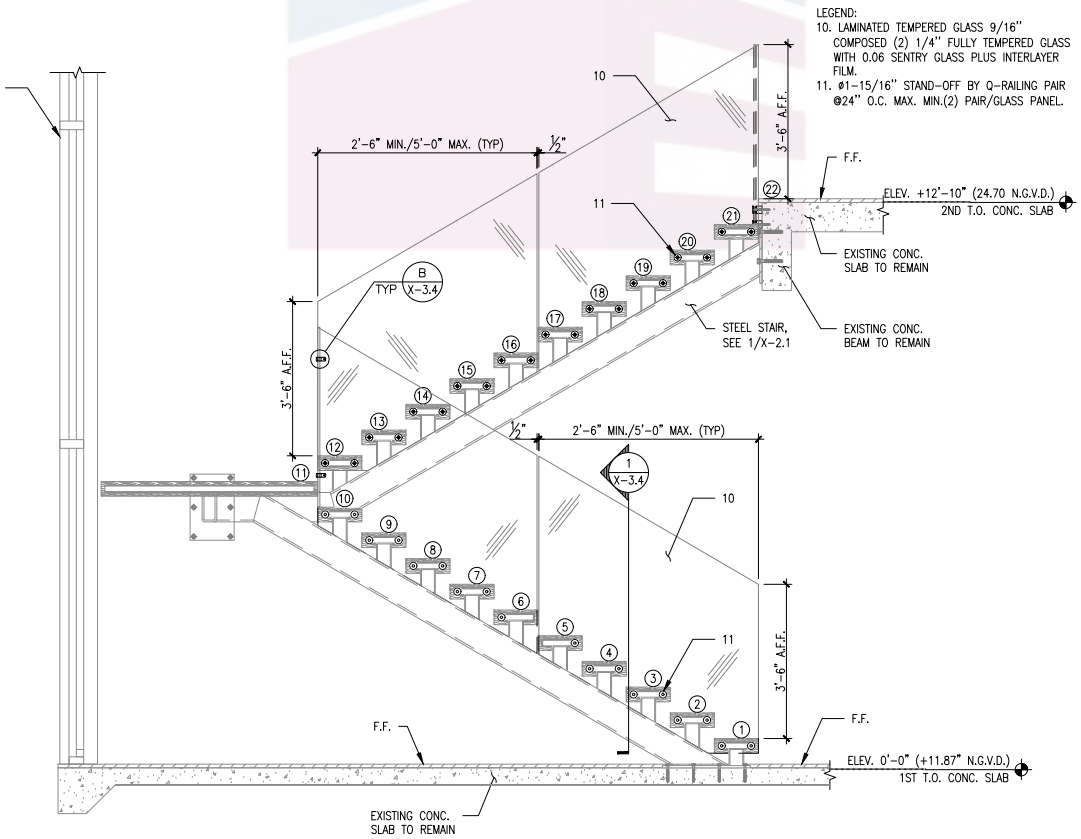


B X-2.0 STEEL STAIR AT SECOND FLOOR PLAN VIEW

SCALE: 3/8"=1'-0"



STAIR SKETCH

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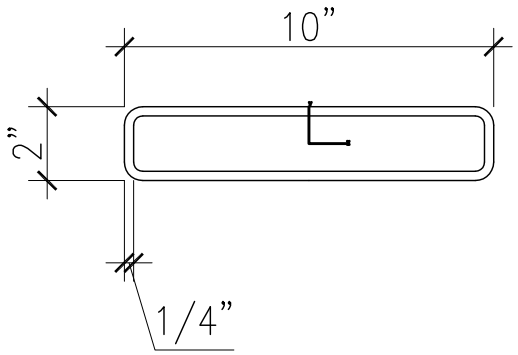
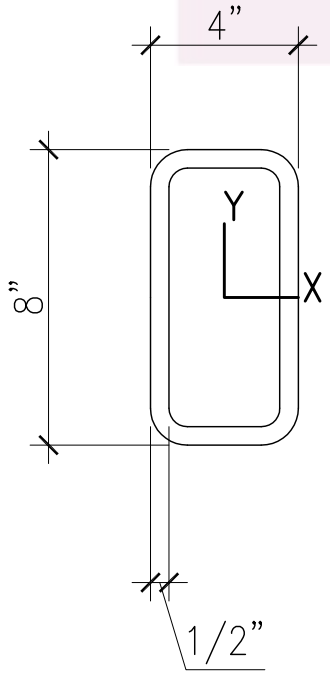
SECTION PROPERTIES



TREAD HSS10X2X1/4
Fy=46KSI

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



STRINGER HSS8X4X1/2
Fy=46KSI

Area = 10.3562 in^2
Ixx = 75.1087 in^4
Iyy = 24.5925 in^4
Jz = 99.7012 in^4
Pxy = −0.0000 in^4
rxx = 2.6931 in
ryy = 1.5410 in
Sxx max = 18.7772 in^3
Sxx min = 18.7772 in^3
Syy max = 12.2963 in^3
Syy min = 12.2963 in^3

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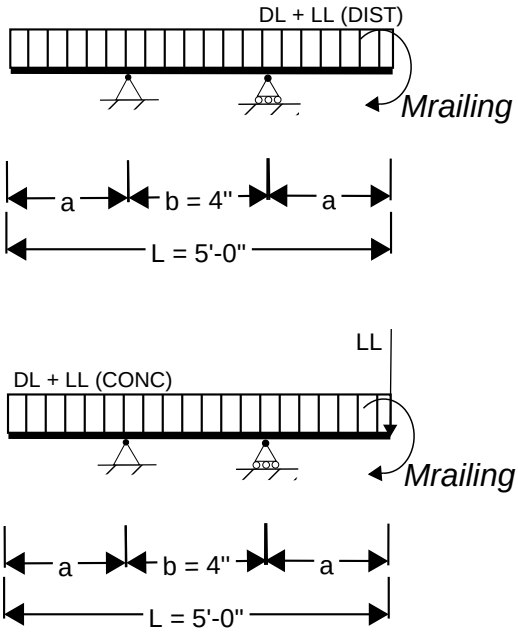
PROJ. NAME:_____

PROJ. NAME:_____ DATE:_____

DESIGNED BY:_____ PAGE:_____

REVIEWED BY:_____

TREAD DESIGN (HSS10X2X1/4)



Loads:

Dead Loads:

Treads = 15psf
Superimposed = 15psf
Railings = 15psf

Live Loads:

LL (dist) = 40psf (See Sheet S-001, December 12, 2023)
LL (conc) = 300#
M (railing) = 200# * 3.5ft = 700#-ft

Loads Combinations:
1- DL + LL (dist) + M (railing)
2- DL + LL (conc) + M (railing)

Loads Processing:
L = 5.0 ft (Worst case) Landing Tread
Tributary Strip = 12in = 1.0ft

q (DL) = 45psf * 1.0 ft = 45 #/ft
q (LL dist) = 40psf * 1.0ft = 40 #/ft

***For tread design see spreadsheet printout**

1)- DL + LL (dist) → M_1 = 0.931 k-ft
2)- DL + LL (conc) → M_2 = 1.522 k-ft (Worst Case)

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Tread analysis V01.ec6

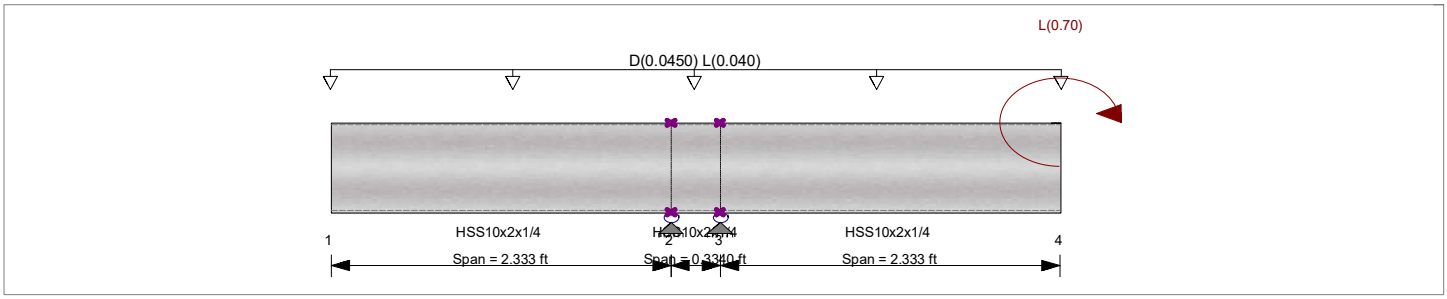
DESCRIPTION: Tread Comb. 1

CODE REFERENCES

Calculations per AISC 360-16, 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 = 0.0450, L = 0.040 k/ft
Moment Load : L = 0.05833 k-ft, Starting at : 5.0 ft and placed every 0.0 ft thereafter

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.117 : 1	Maximum Shear Stress Ratio =	0.211 : 1
Section used for this span	HSS10x2x1/4	Section used for this span	HSS10x2x1/4
Ma : Applied	0.931 k-ft	Va : Applied	2.110 k
Mn / Omega : Allowable	7.947 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.334 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.037 in	Ratio =	1,523	>=360	Span: 3 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.040 in	Ratio =	1392	>=360.	Span: 3 : +D+L
Max Upward Total Deflection	-0.000 in	Ratio =	29889	>=360.	Span: 2 : +D+L

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+L	0.0081	0.000		0.0000	0.000
2		0.0000	0.000	+D+L	-0.0001	0.185
3	+D+L	0.0402	2.333		0.0000	0.185

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		0.113	2.308	
Max Upward from Load Combinations		0.068	2.308	
Max Upward from Load Cases		0.113	2.196	
Max Downward from all Load Conditions (Resisting Uplift)		-1.996		
Max Downward from Load Combinations (Resisting Uplift)		-1.883		
Max Downward from Load Cases (Resisting Uplift)		-1.996		
D Only		0.113	0.113	
+D+L		-1.883	2.308	
+D+0.750L		-1.384	1.759	
+0.60D		0.068	0.068	
L Only		-1.996	2.196	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Tread analysis V01.ec6

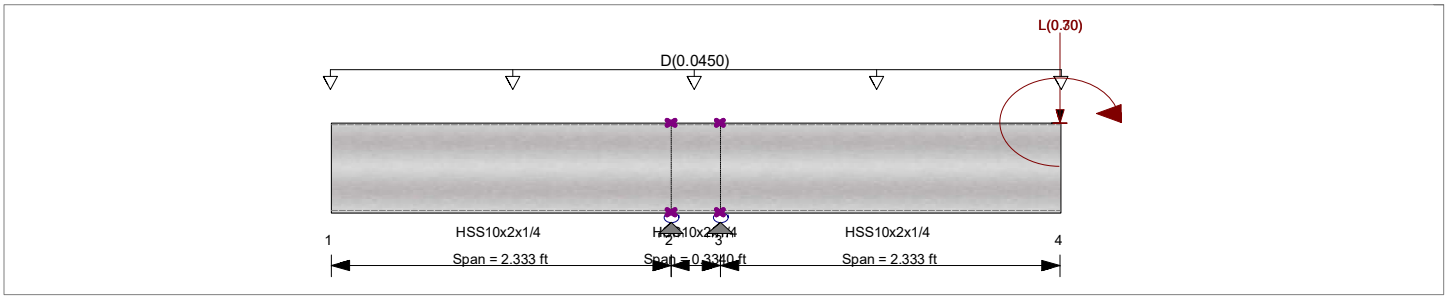
DESCRIPTION: Tread Comb. 2

CODE REFERENCES

Calculations per AISC 360-16, 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 = 0.0450 k/ft
Moment Load : L = 0.05833 k-ft, Starting at : 5.0 ft and placed every 0.0 ft thereafter
Load(s) for Span Number 3
Point Load : L = 0.30 k @ 2.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.192 : 1	Maximum Shear Stress Ratio =	0.419 : 1
Section used for this span	HSS10x2x1/4	Section used for this span	HSS10x2x1/4
Ma : Applied	1.522 k-ft	Va : Applied	4.199 k
Mn / Omega : Allowable	7.947 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.334 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.057 in	Ratio =	979 >=360	Span: 3 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.061 in	Ratio =	924 >=360.	Span: 3 : +D+L
Max Upward Total Deflection	-0.000 in	Ratio =	20933 >=360.	Span: 2 : +D+L

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+L	0.0064	0.000		0.0000	0.000
2		0.0000	0.000	+D+L	-0.0002	0.191
3	+D+L	0.0606	2.333		0.0000	0.191

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.113	4.604		
Max Upward from Load Combinations	0.068	4.604		
Max Upward from Load Cases	0.113	4.491		
Max Downward from all Load Conditions (Resisting Uplift)	-4.191			
Max Downward from Load Combinations (Resisting Uplift)	-4.079			
Max Downward from Load Cases (Resisting Uplift)	-4.191			
D Only	0.113	0.113		
+D+L	-4.079	4.604		
+D+0.750L	-3.031	3.481		
+0.60D	0.068	0.068		
L Only	-4.191	4.491		

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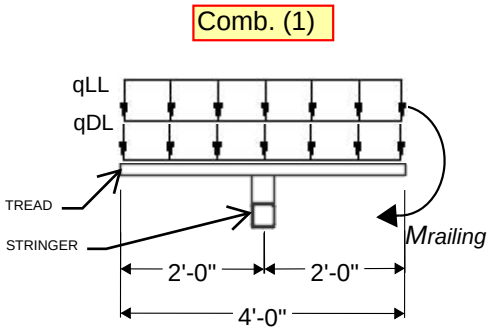
STRINGER DESIGN (HSS8X4X3/8)

Loads:

DL = 45 psf
LL = 40 psf (dist.) or P_{LL}= 300# (conc.)
Mrailing = 200# * 3.5 ft= 700#-ft

Loads Combinations:

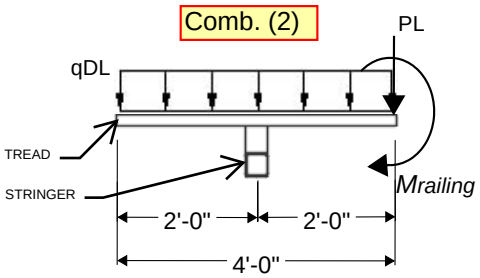
(1) DL+LL (distributed) stringer fully loaded + Railing Action
(2) DL+LL (concentrated) stringer fully loaded + Railing Action
(3) DL+LL stringer partially loaded + Railing Action



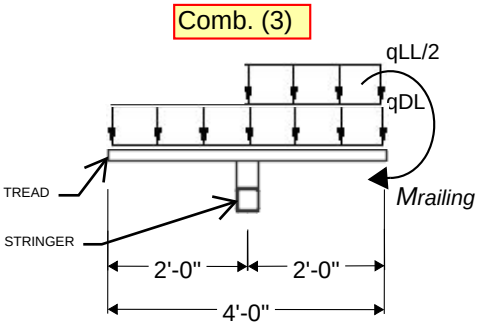
Loads Processing:

For Stringers

<u>Tributary Strips:</u>
- For DL in all combinations = 4.0 ft
- For LL in Comb. 1 = 4.0 ft
- For LL in Comb. 3 = 2.0 ft



qDL= 45 psf x4.0 ft = 180 #/ft
qLL full= 40 psf x 4.0 ft = 160 #/ft
qLL partial= 40 psf x2.0 ft = 80 #/ft
MLL =40psf x 2.0ft x 2.0ft/2 =80.0 #-ft/ft



For Landing Stringers

<u>Tributary Strips:</u>
- For DL in all combinations = 5.0 ft
- For LL in Comb. 1 = 5.0 ft
- For LL in Comb. 3 = 2.5 ft

qDL= 45 psf x5.0 ft = 225 #/ft
qLL full= 40 psf x 5.0 ft = 200 #/ft
qLL partial= 40 psf x2.5 ft = 100 #/ft
MLL =40psf x 2.5ft x 2.5ft/2 =125 #-ft/ft

*For stringers design and analysis see RISA Report printout.

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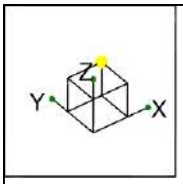
GRAPHICS
Software - Risa-3D
v.22.0.1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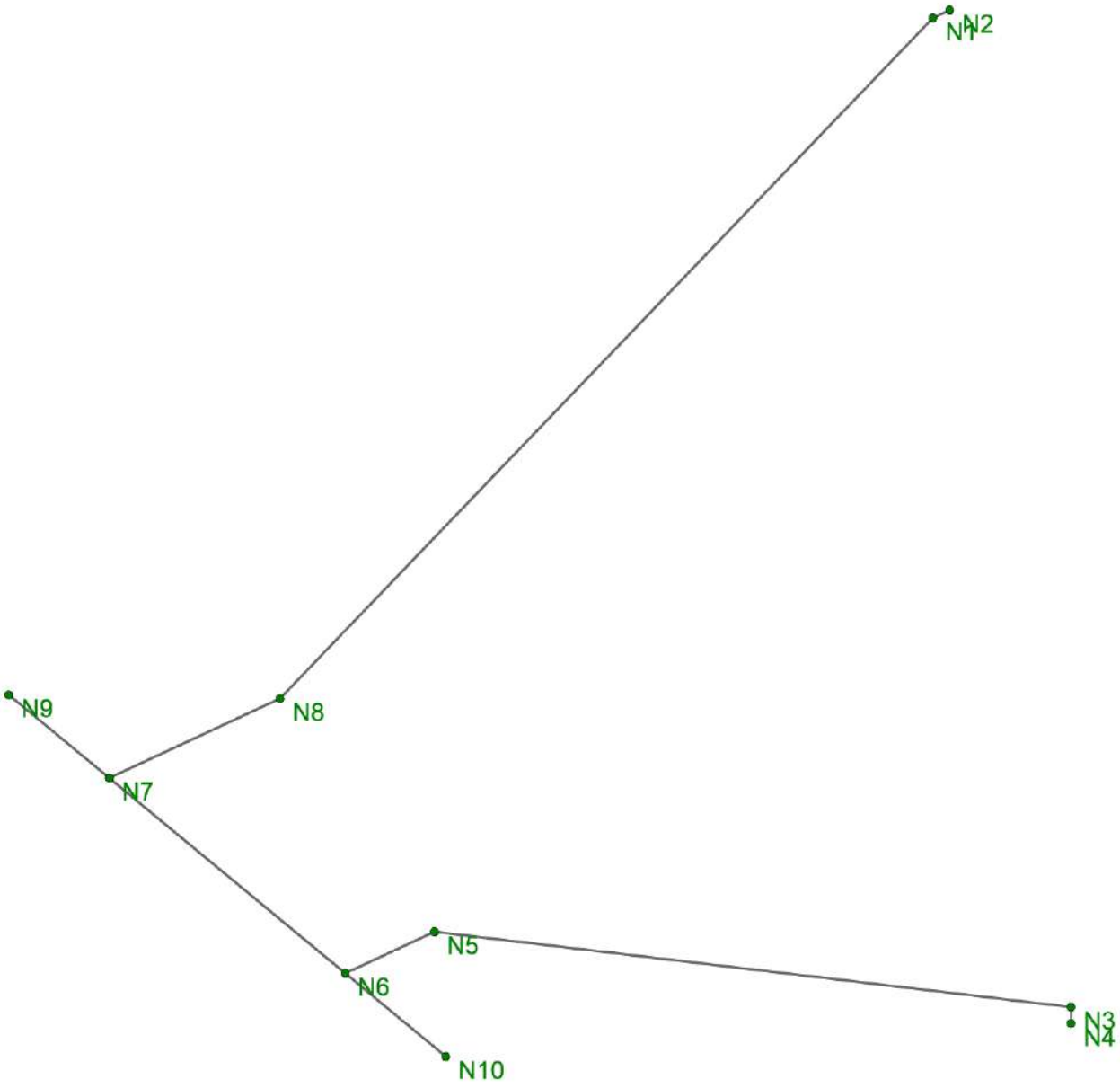
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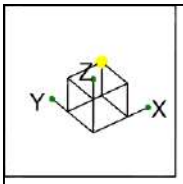
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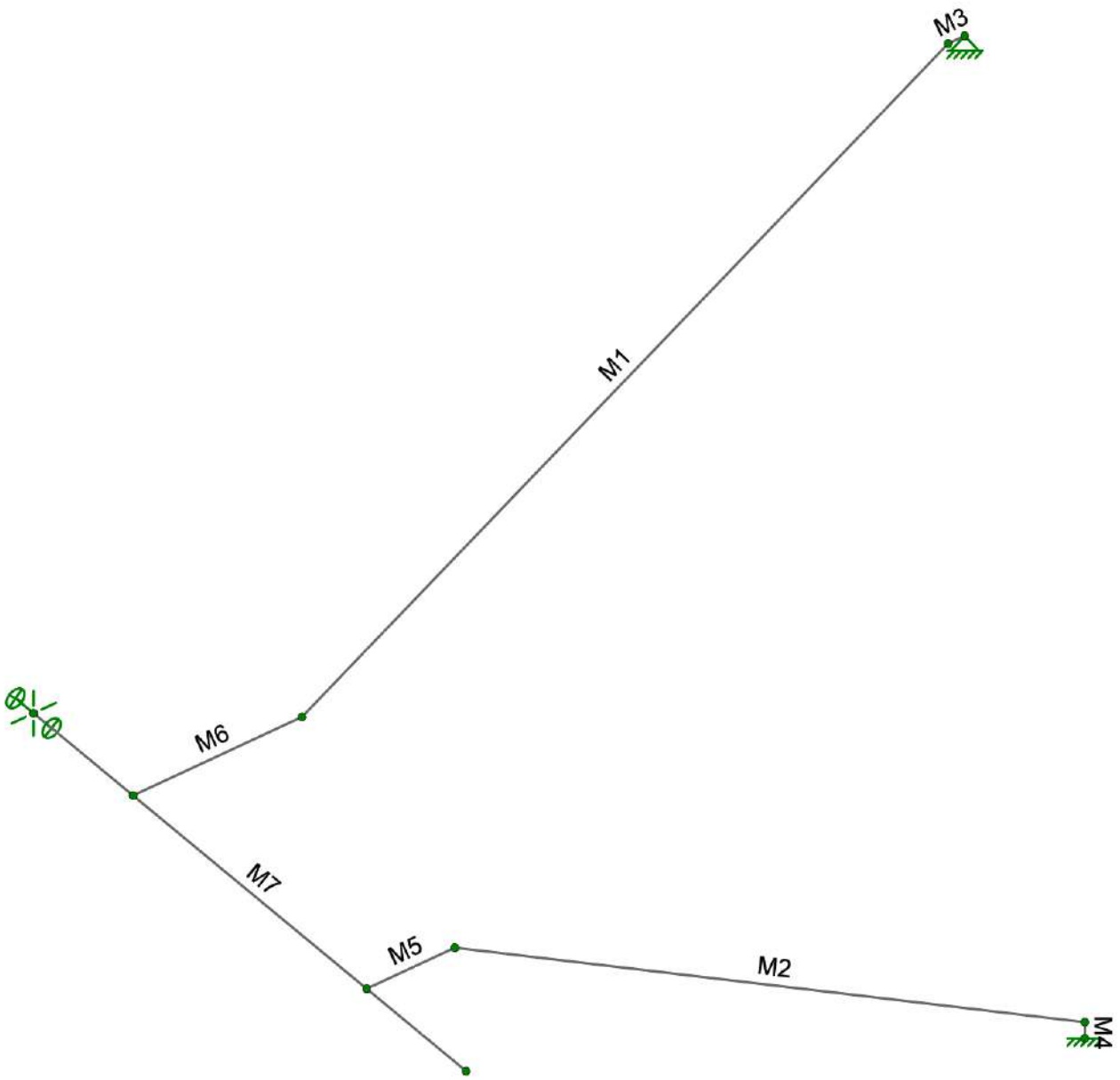
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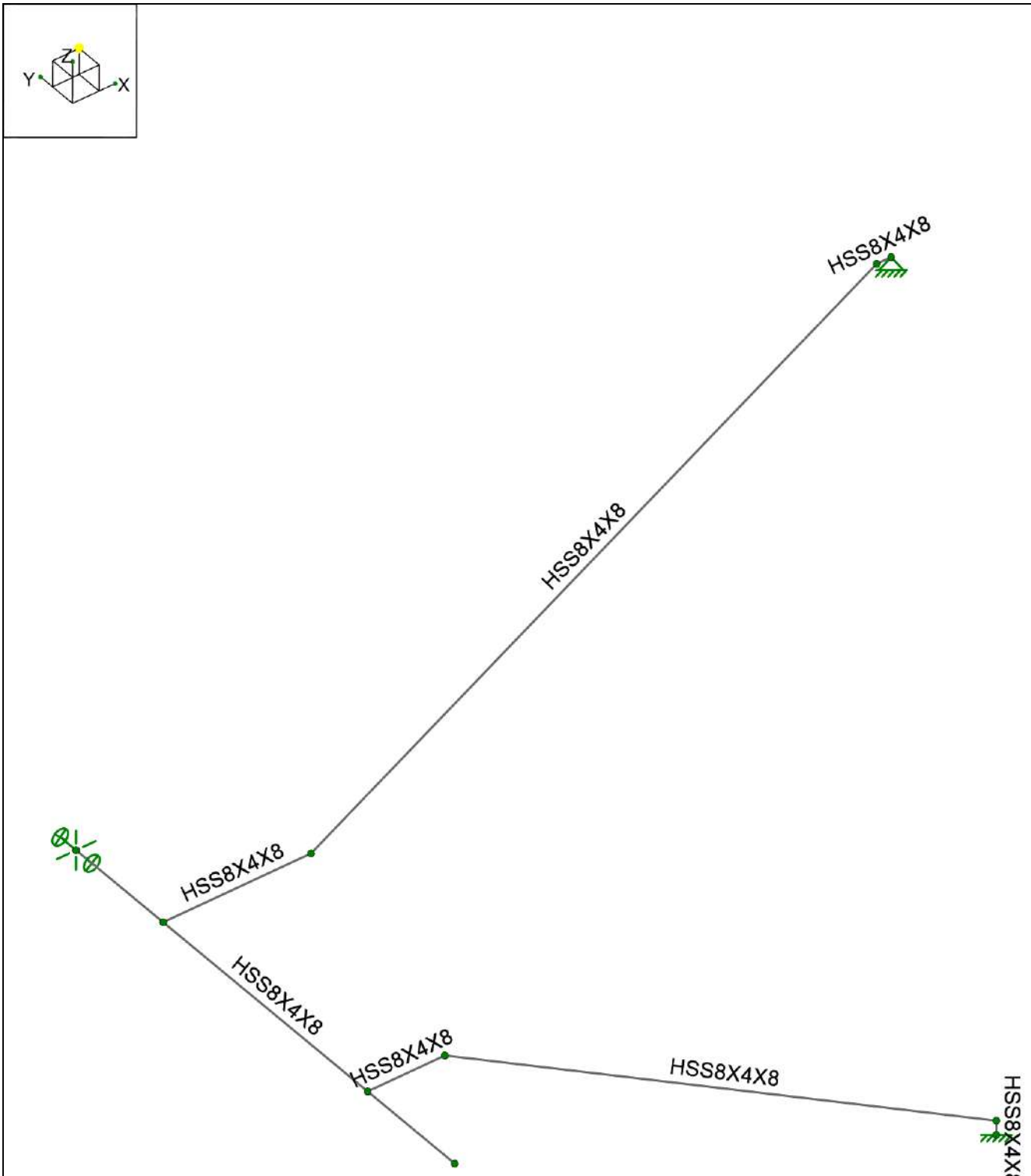
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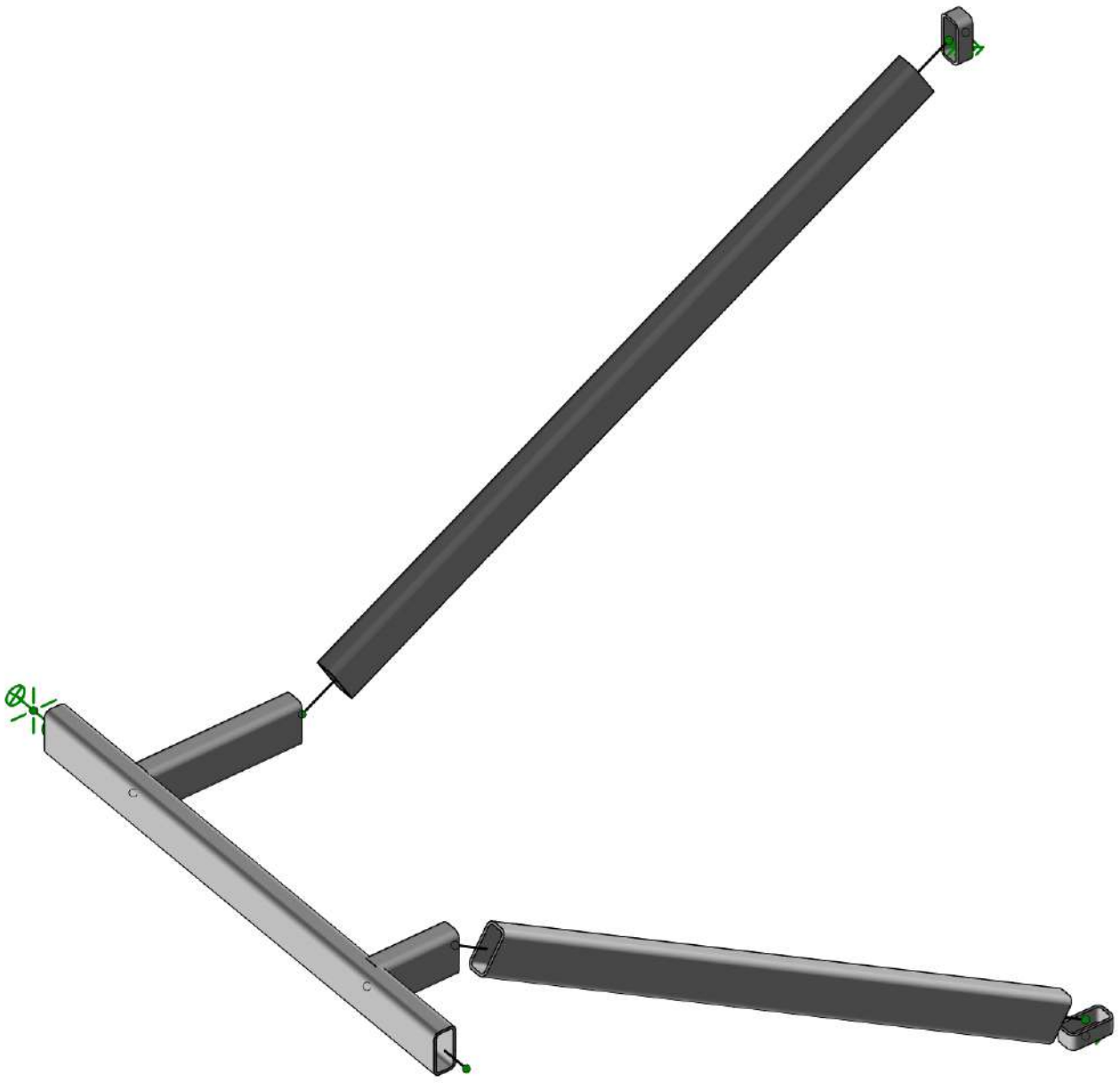
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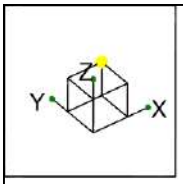
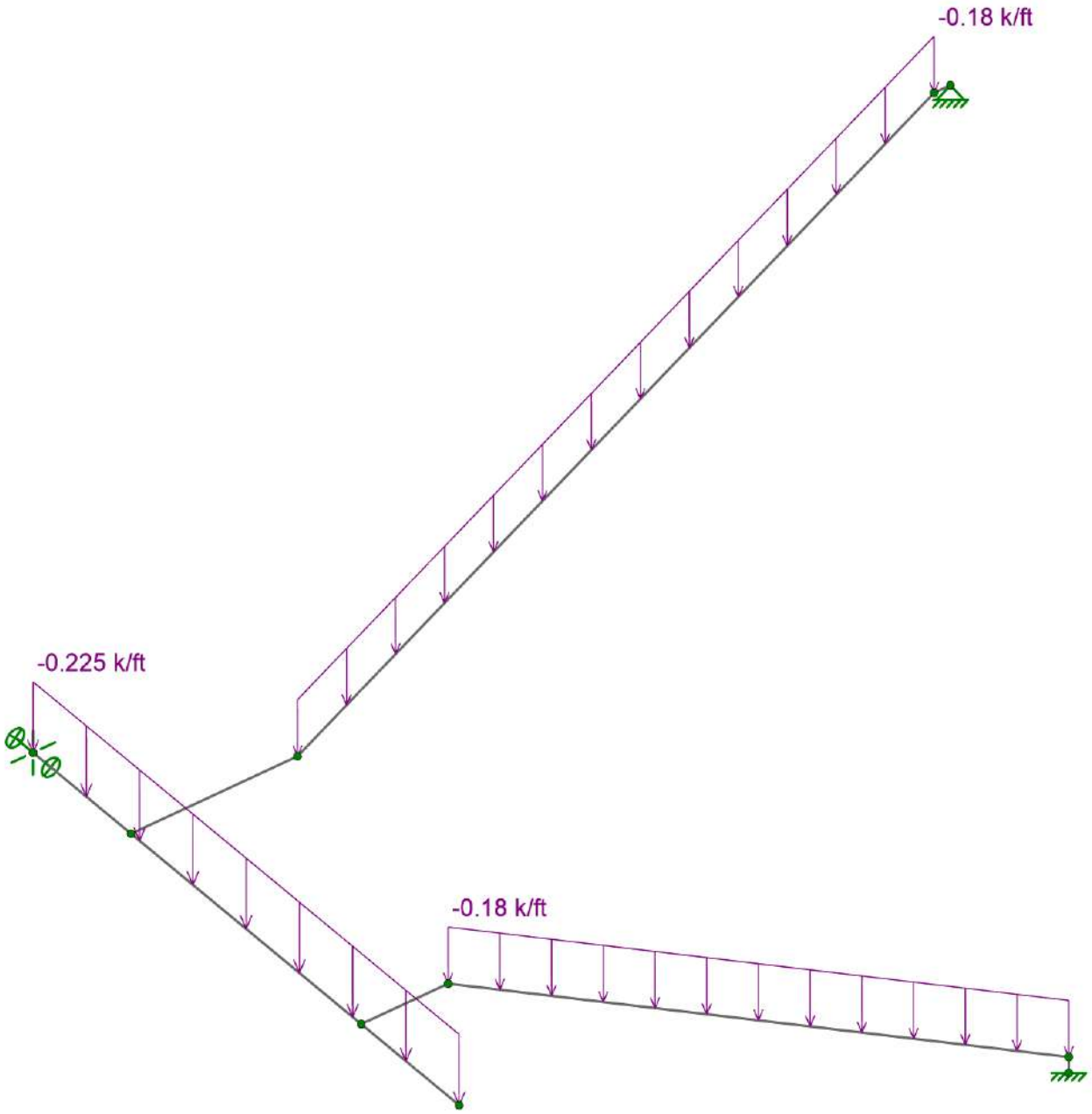
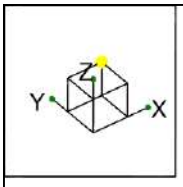
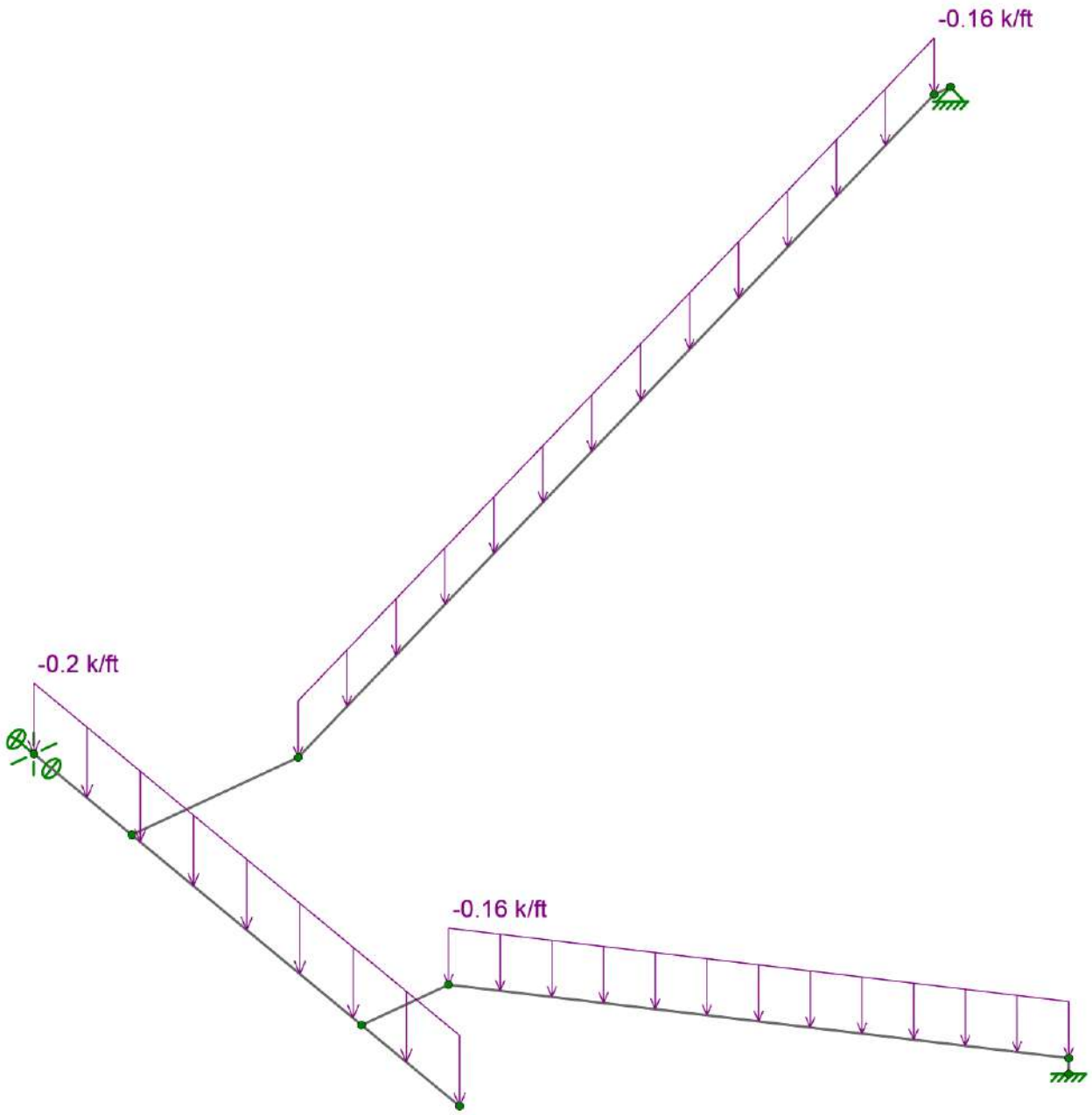




Loads: BLC 2, D.L

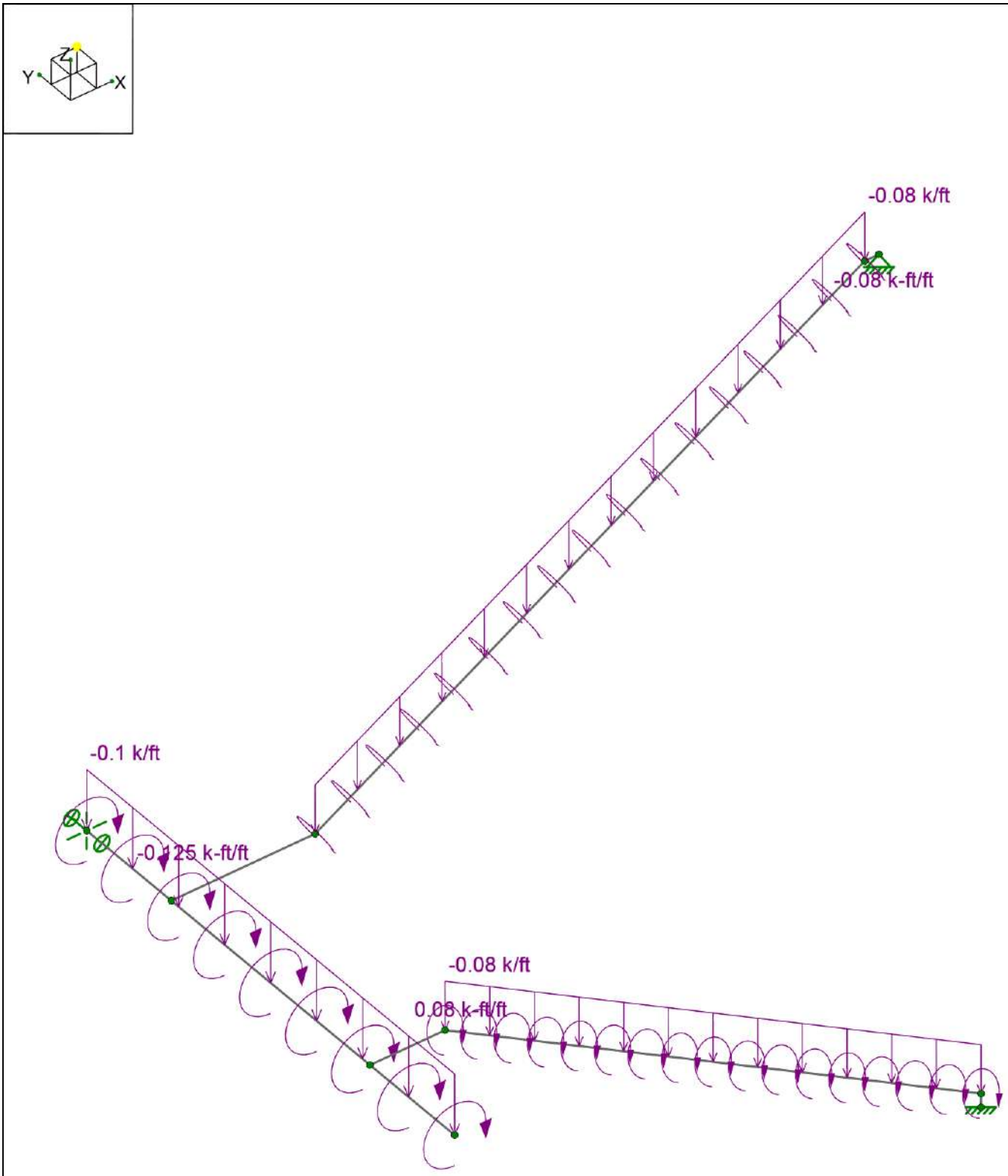
RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-8
	MauricioMenendez		Jun 30, 2025 at 04:55 PM
			Stair_V01.r3d





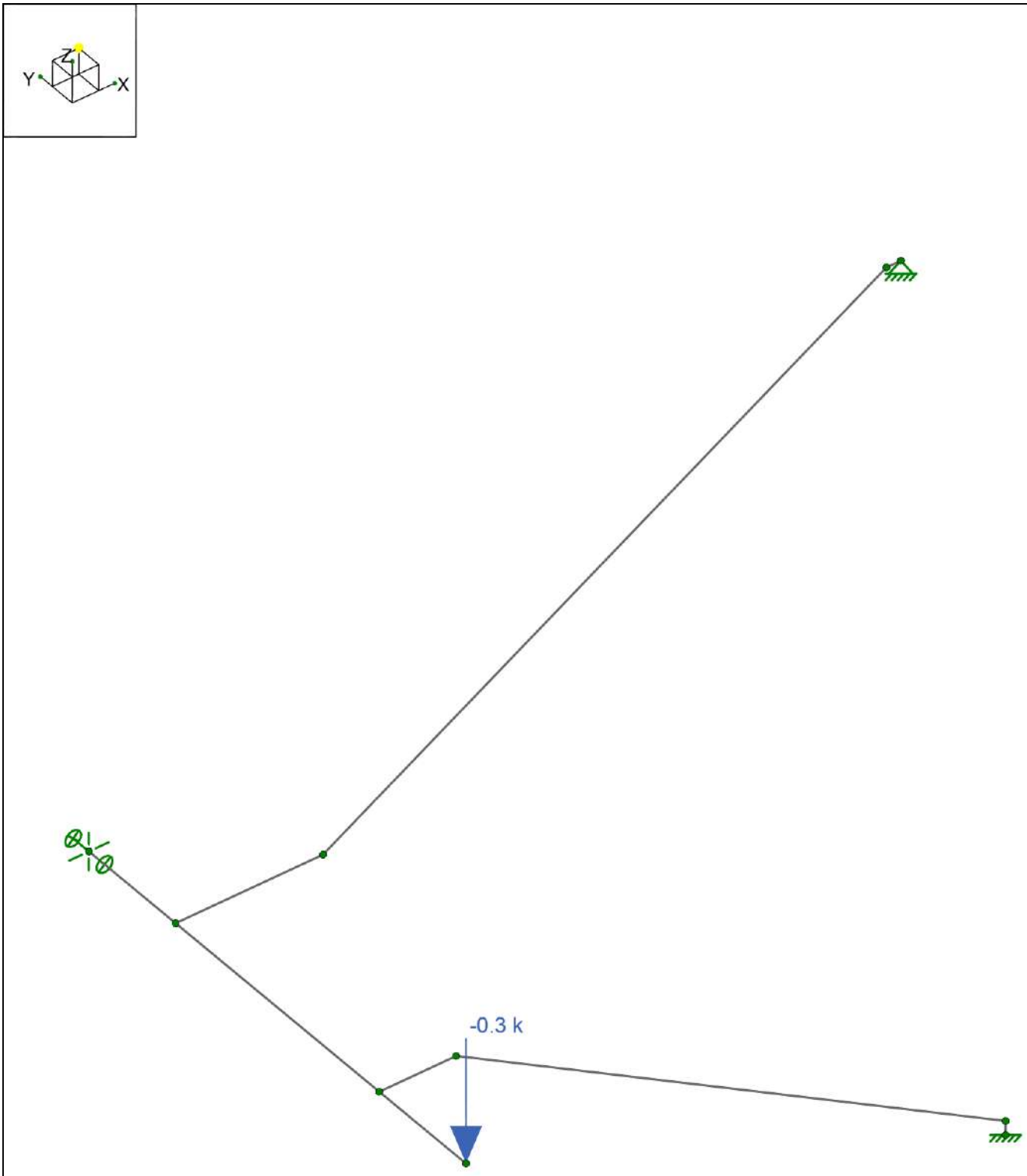
Loads: BLC 3, L.L (FULL)

RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-9
	MauricioMenendez		Jun 30, 2025 at 04:56 PM
			Stair_V01.r3d



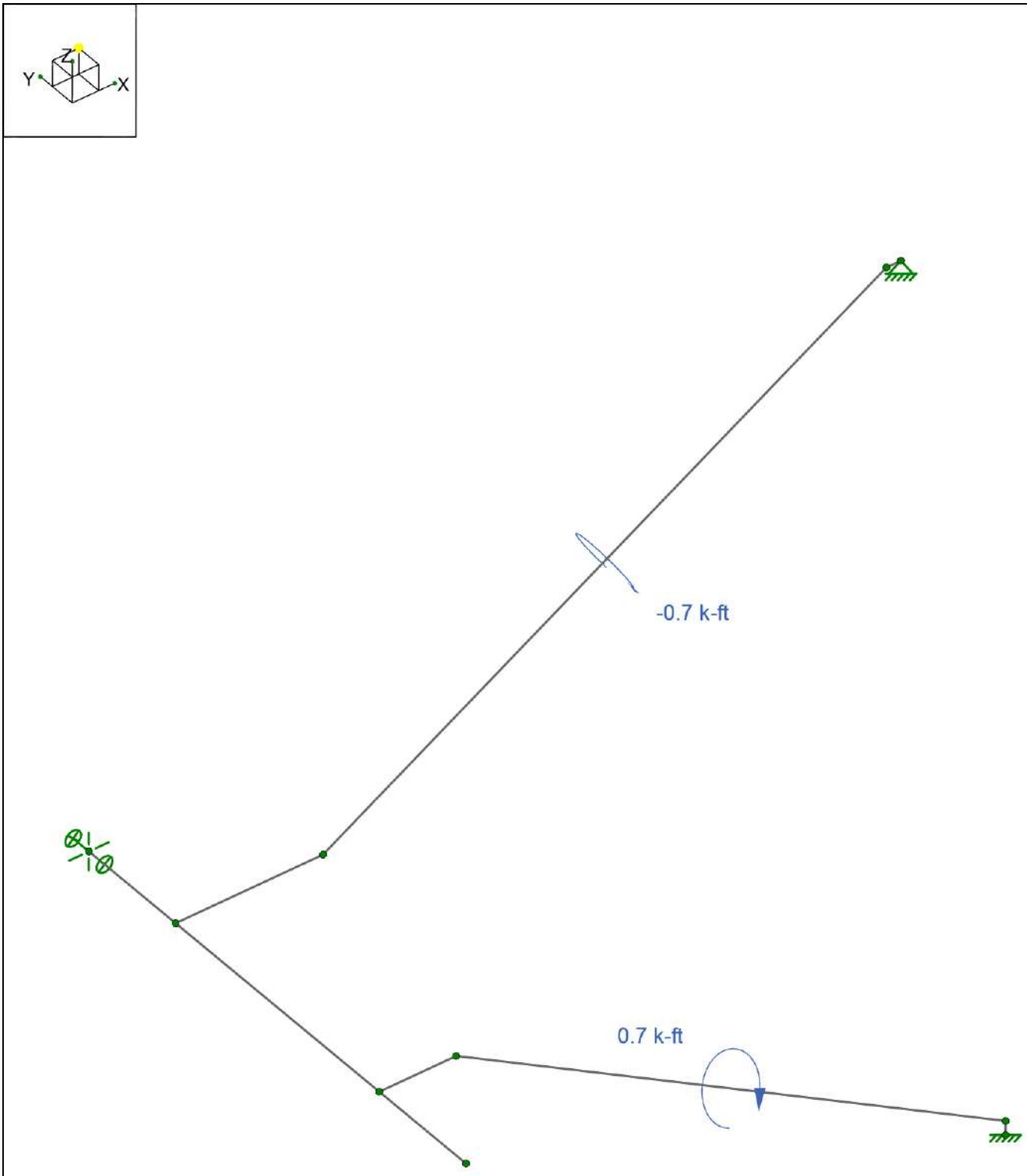
Loads: BLC 4, L.L (PARTIAL)

RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-10
	MauricioMenendez		Jun 30, 2025 at 04:57 PM
			Stair_V01.r3d



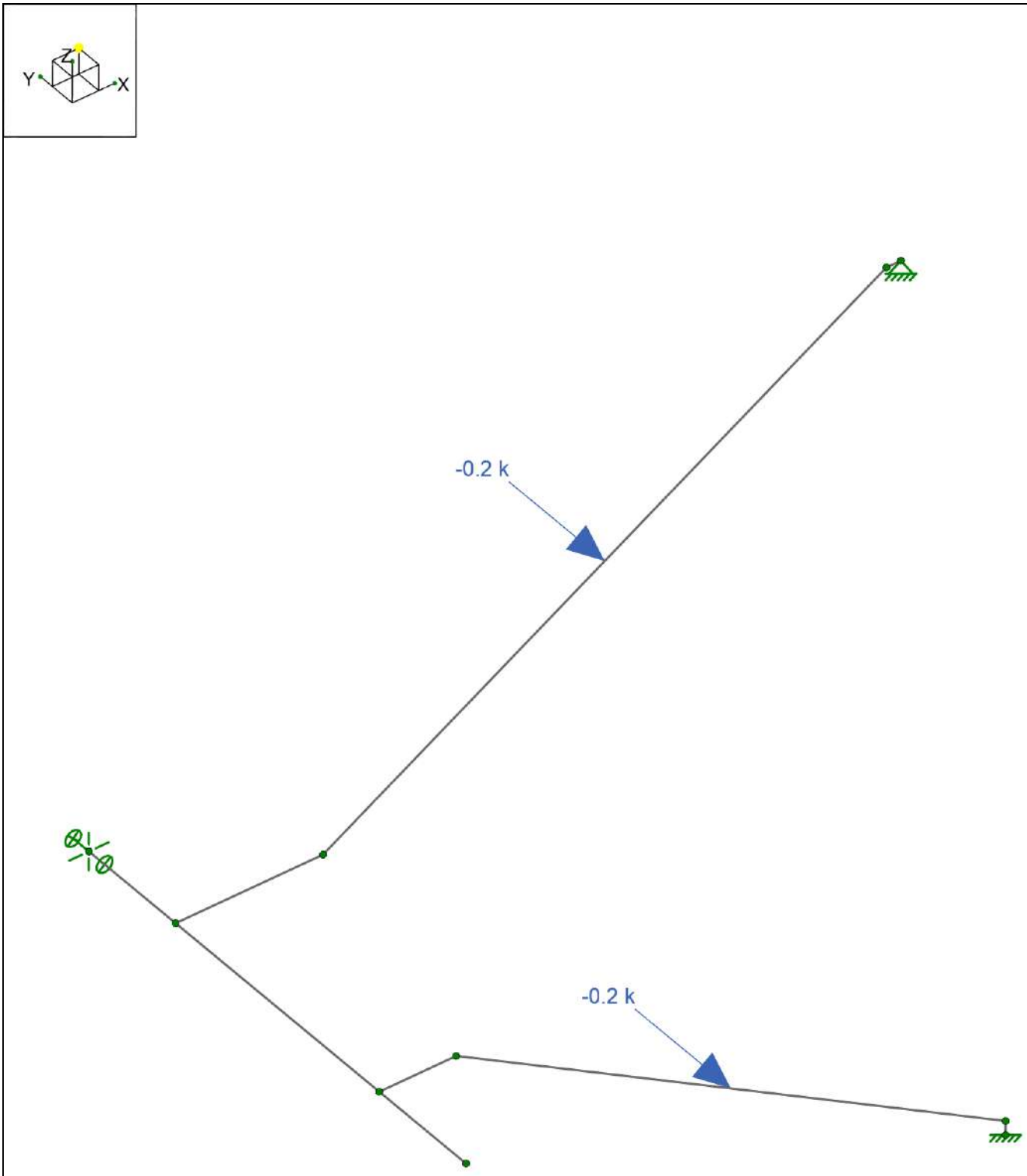
Loads: BLC 5, L.L (CONC)

IRISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-11
	MauricioMenendez		Jun 30, 2025 at 04:57 PM
			Stair_V01.r3d



Loads: BLC 6, M (RAILING)

RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-12
	MauricioMenendez		Jun 30, 2025 at 05:18 PM
			Stair_V01.r3d



Loads: BLC 7, R (RAILING)

RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-13
	MauricioMenendez		Jun 30, 2025 at 05:18 PM
			Stair_V01.r3d

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SOFTWARE:
RISA 3D v.22.0.1
INPUT

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	11.552	6.697248	11.654902	
2	N2	11.802	6.697248	11.654902	
3	N3	10.089704	2	0.25	
4	N4	10.089704	2	0	
5	N5	0.586428	2	5.915728	
6	N6	-0.744	2	5.915728	
7	N7	-0.744	6.697248	5.915728	
8	N8	1.806173	6.697248	5.915728	
9	N9	-0.744	8.697247	5.915728	
10	N10	-0.744	0	5.915728	

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	N4	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
2	N2	Reaction	Reaction	Reaction			
3	N9	Reaction	Reaction	Reaction		Reaction	

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	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	STRINGER	HSS8X4X8	Beam	Tube	A500 Gr.B RECT	Typical	9.74	23.6	71.8	61.1
2	START STRINGER	HSS8X4X8	Column	Tube	A500 Gr.B RECT	Typical	9.74	23.6	71.8	61.1

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N8		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical
2	M2	N5	N3		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical
3	M3	N1	N2		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical
4	M4	N3	N4	90	START STRINGER	Column	Tube	A500 Gr.B RECT	Typical
5	M5	N5	N6		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical
6	M6	N7	N8		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical
7	M7	N9	N10		STRINGER	Beam	Tube	A500 Gr.B RECT	Typical

Basic Load Cases

	BLC Description	Category	Z Gravity	Point	Distributed
1	SELF WIEGHT	DL	-1		
2	D.L	DL			3
3	L.L (FULL)	LL			3
4	L.L (PARTIAL)	LL			6
5	L.L (CONC)	LL		1	
6	M (RAILING)	LL		2	
7	R (RAILING)	LL		2	

Member Distributed Loads (BLC 2 : D.L)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	-0.18	-0.18	0	%100
2	M7	Z	-0.225	-0.225	0	%100
3	M2	Z	-0.18	-0.18	0	%100

Member Distributed Loads (BLC 3 : L.L (FULL))

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Z	-0.16	-0.16	0	%100
2	M7	Z	-0.2	-0.2	0	%100
3	M2	Z	-0.16	-0.16	0	%100

Member Distributed Loads (BLC 4 : L.L (PARTIAL))

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Mx	-0.08	-0.08	0	%100
2	M2	Mx	0.08	0.08	0	%100
3	M7	Mx	-0.125	-0.125	0	%100
4	M1	Z	-0.08	-0.08	0	%100
5	M2	Z	-0.08	-0.08	0	%100
6	M7	Z	-0.1	-0.1	0	%100

Member Point Loads (BLC 5 : L.L (CONC))

	Member Label	Direction	Magnitude [k, k-ft]	Location [(ft, %)]
1	M7	Z	-0.3	%100

Member Point Loads (BLC 6 : M (RAILING))

	Member Label	Direction	Magnitude [k, k-ft]	Location [(ft, %)]
1	M1	Mx	-0.7	%50
2	M2	Mx	0.7	%50

Member Point Loads (BLC 7 : R (RAILING))

	Member Label	Direction	Magnitude [k, k-ft]	Location [(ft, %)]
1	M1	Y	-0.2	%50
2	M2	Y	-0.2	%50

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	D.L+L.L (FULL)+M (RAILING)+R (RAILING)	Yes	Y	DL	1	3	1	6	1	7	1
2	D.L+L.L (CONC)+M (RAILING)+R (RAILING)	Yes	Y	DL	1	5	1	6	1	7	1
3	D.L+L.L (PARTIAL)+M (RAILING)+R (RAILING)	Yes	Y	DL	1	4	1	6	1	7	1

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REPORTS

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	N4	max	-2.406	2	0.527	1	5.885	1	-3.005	2	14.265	1	0.046	3
2		min	-3.675	1	0.384	2	3.675	2	-4.124	1	9.241	2	-0.181	1
3	N2	max	9.444	1	0.232	1	6.915	1	0	3	0	3	0	3
4		min	6.206	2	0.185	2	4.359	2	0	1	0	1	0	1
5	N9	max	-3.8	2	-0.169	2	-0.232	1	0	3	3.276	1	0	3
6		min	-5.769	1	-0.358	1	-0.486	2	0	1	2.403	3	0	1
7	Totals:	max	0	2	0.4	2	12.567	1						
8		min	0	3	0.4	3	7.548	2						

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	-0.009	2	-0.004	2	5.654e-3	3	-1.156e-3	2	4.279e-3	3
2		min	0	1	-0.013	3	-0.006	1	3.593e-3	2	-2.055e-3	1	3.096e-3	2
3	N2	max	0	2	0	2	0	2	5.654e-3	3	-1.168e-3	2	4.28e-3	3
4		min	0	1	0	1	0	1	3.593e-3	2	-2.073e-3	1	3.098e-3	2
5	N3	max	0	2	0	2	0	2	2.668e-4	1	-2.062e-4	2	9.586e-6	1
6		min	0	1	0	1	0	1	1.944e-4	2	-3.182e-4	1	-2.438e-6	3
7	N4	max	0	1	0	2	0	2	0	1	0	2	0	1
8		min	0	2	0	1	0	1	0	2	0	1	0	3
9	N5	max	-0.163	2	-0.059	2	-0.278	2	5.569e-3	1	-3.023e-3	2	-3.277e-3	2
10		min	-0.238	1	-0.086	1	-0.405	1	3.974e-3	2	-4.237e-3	1	-4.729e-3	1
11	N6	max	-0.163	2	0	1	-0.325	2	5.835e-3	1	-2.906e-3	2	-4.115e-3	2
12		min	-0.238	1	0	2	-0.472	1	4.166e-3	2	-4.059e-3	1	-6.045e-3	1
13	N7	max	0.047	1	0	1	-0.094	2	5.869e-3	1	-1.06e-3	2	-1.501e-4	3
14		min	0.029	2	0	2	-0.139	1	3.99e-3	2	-1.385e-3	1	-2.636e-4	1
15	N8	max	0.048	1	0.014	3	-0.064	2	5.778e-3	1	-6.385e-4	3	1.055e-3	3
16		min	0.03	2	0.005	2	-0.101	1	4.048e-3	2	-7.883e-4	1	5.835e-4	2
17	N9	max	0	1	0	1	0	2	5.777e-3	1	0	3	2.771e-3	1
18		min	0	2	0	2	0	1	3.877e-3	2	0	1	1.767e-3	2
19	N10	max	-0.262	2	0	1	-0.427	2	5.888e-3	1	-2.906e-3	2	-4.115e-3	2
20		min	-0.383	1	0	2	-0.613	1	4.248e-3	2	-4.059e-3	1	-6.045e-3	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	M1	HSS8X4X8	0.177	11.31	1	0.067	11.31	y	3	160.881	268.287	32.824	53.942	1.944	H1-1b
2	M2	HSS8X4X8	0.367	11.064	1	0.129	11.064	y	1	164.462	268.287	32.824	53.942	2.668	H1-1b
3	M3	HSS8X4X8	0.051	0	1	0.068	0.25	y	1	268.22	268.287	32.824	53.942	1.666	H1-1b
4	M4	HSS8X4X8	0.414	0	1	0.042	0.25	y	1	268.22	268.287	32.824	53.942	1.025	H1-1b
5	M5	HSS8X4X8	0.177	1.33	1	0.045	0	y	1	266.396	268.287	32.824	53.942	1.51	H1-1b
6	M6	HSS8X4X8	0.181	2.55	1	0.046	2.55	y	3	261.402	268.287	32.824	53.942	1.792	H1-1b
7	M7	HSS8X4X8	0.424	2.084	1	0.24	1.993	z	1	198.276	268.287	32.824	53.942	1.793	H1-1b

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STAIR CONNECTIONS DESIGN

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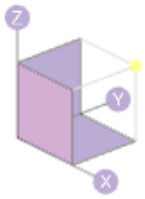
BOTTOM CONNECTION DESIGN (NODE N4)

LOADS (RISA REACTIONS):

ASD:

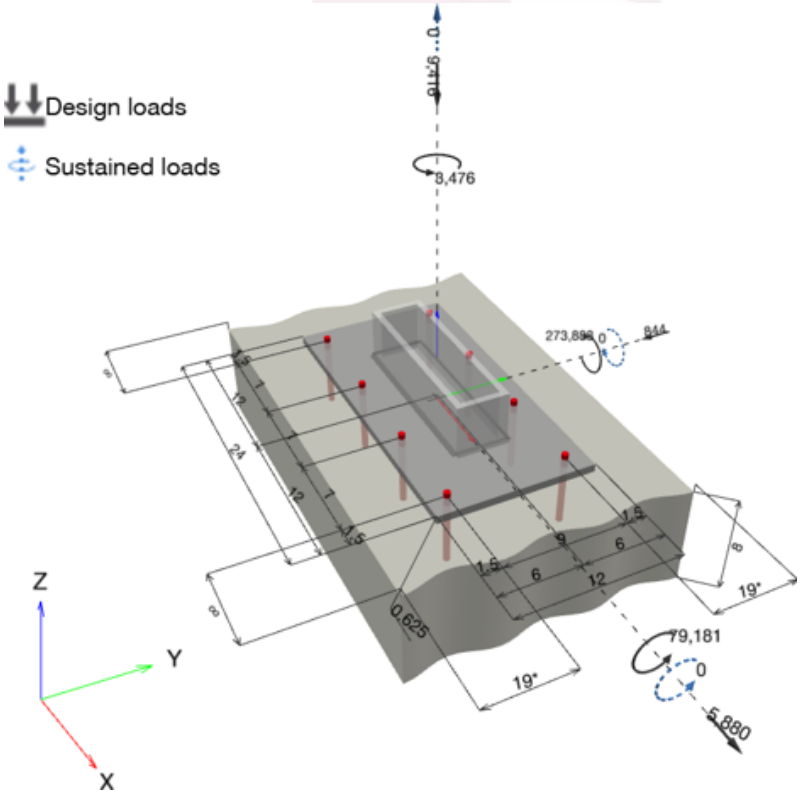


Rx= 3675 lb
Ry= 527 lb
Rz= 5885 lb
Mx= 49488 lb-in
My= 171180 lb-in
Mz= 2172 lb-in

LOADS PROCESSING (ASD TO LRFD):

LRFD:
Rx= 3675 lb x 1.60 = 5880 lb ("Rx" HILTI) "Conservative"
Ry= 527 lb x 1.60 = 844 lb ("Ry" HILTI) "Conservative"
Rz= 5885 lb lb x 1.60 = 9416 lb ("Rz" HILTI) "Conservative"
Mx= 49488 lb-in x 1.60 = 79181 lb-in ("Mx" HILTI) "Conservative"
My= 171180 lb-in x 1.60 = 273888 lb-in ("My" HILTI) "Conservative"
Mz= 2172 lb-in x 1.60 = 3476 lb-in ("Mz" HILTI) "Conservative"

* For anchor design see spreadsheet printout



CONN. SKETCH

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	BOTTOM CONNECTION_V01	Date:	7/18/2025
Fastening point:			

Specifier's comments:

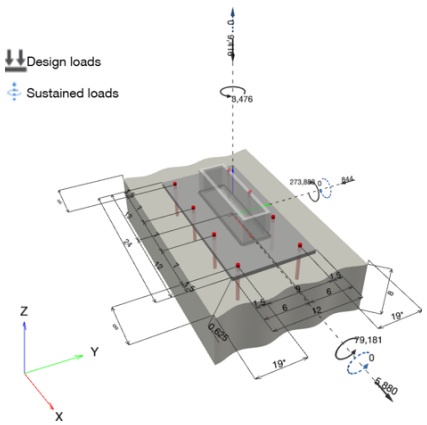
1 Anchor Design

1.1 Input data

Anchor type and diameter:	**KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2**
Item number:	418077 KH-EZ 1/2"x6" (element) / 2362252 KHC 1/2" LARGE (capsule)
Specification text:	Hilti ∅ 1/2 in KWIK-X (Carbon Steel) with KH-EZ with KHC 1/2" LARGE with 5 in nominal embedment depth per ICC-ES ESR-5065, Hammer drill bit installation per MPII,
Effective embedment depth:	$h_{ef,act}$ = 5.000 in. ($h_{ef,limit}$ = - in.), h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-5065
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-19 / Chem
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.625 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 12.000 in. x 0.625 in.;
Profile:	Rectangular HSS (AISC), HSS14X4X.500; (L x W x T) = 14.000 in. x 4.000 in. x 0.500 in.
Base material:	uncracked concrete, 3000, f_c' = 3,000 psi; h = 8.000 in., Temp. short/long: 32/32 °F
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

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

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



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	BOTTOM CONNECTION_V01	Date:	7/18/2025
Fastening point:			

1.1.1 Design results

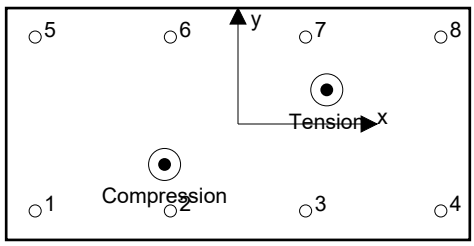
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -9,416; V_x = 5,880; V_y = -844; M_x = 79,181; M_y = -273,888; M_z = 3,476;	no	99

1.2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	469	731	713	-160
2	0	767	756	-128
3	2,627	799	796	-67
4	2,751	758	758	-7
5	115	719	704	-148
6	2,470	750	736	-145
7	6,926	748	739	-117
8	3,900	682	679	-71



Resulting tension force in (x/y)=(4.596/1.767): 19,258 [lb]
Resulting compression force in (x/y)=(-3.801/-2.094): 31,425 [lb]

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

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	6,926	11,777	59	OK
Bond Strength**	19,258	24,976	78	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	19,258	20,633	94	OK

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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	BOTTOM CONNECTION_V01	Date:	7/18/2025
Fastening point:			

1.3.1 Steel Strength

N_{sa} = ESR value	refer to ICC-ES ESR-5065
$\phi\ N_{sa} \geq N_{ua}$	ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

N_{sa} [lb]
18,119

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,119	0.650	11,777	6,926

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

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na} \psi_{ec2,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba}$ ACI 318-19 Eq. (17.6.5.1b)

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

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2 c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,197	0.500	5.000	19.000	1.000	2,197

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
4.096	1.125	11.987	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.034	808.92	197.89	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
0.632	0.862	1.000	17,252

Results

N_{ag} [lb]	ϕ_{bond}	ϕN_{ag} [lb]	N_{ua} [lb]
38,424	0.650	24,976	19,258

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

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

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
5.000	4.096	1.125	19.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
11.987	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
864.00	225.00	0.647	0.870	1.000	1.000	14,697

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
31,742	0.650	20,633	19,258

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

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	799	5,547	15	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	5,940	72,542	9	OK
Concrete edge failure in direction y-**	5,940	23,796	25	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-5065
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

V_{sa} [lb]
9,245

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
9,245	0.600	5,547	799

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.2 Pryout Strength (Concrete Breakout Strength controls)

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

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

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

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

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

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	5.000	0.083	0.579	19.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	11.987	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
864.00	225.00	0.989	0.928	1.000	1.000	14,697

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
103,632	0.700	72,542	5,940

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.3 Concrete edge failure in direction y-

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

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

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

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

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

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

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

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
19.000	-	0.460	1.400	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
4.000	1.000	0.500	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
624.00	1,624.50	0.984	1.000	1.887	34,032

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
33,995	0.700	23,796	5,940

*Anchor row defined by: Anchor 1-4; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.933	0.250	1.000	99	OK

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

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

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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1.7 Installation data

Anchor type and diameter: KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2

Profile: Rectangular HSS (AISC), HSS14X4X.500; (L x W x T) = 14.000 in. x 4.000 in. x 0.500 in.

Item number: 418077 KH-EZ 1/2"x6" (element) / 2362252 KHC 1/2" LARGE (capsule)

Hole diameter in the fixture: d_f = 0.625 in.

Maximum installation torque: -

Plate thickness (input): 0.625 in.

Hole diameter in the base material: 0.500 in.

Hole depth in the base material: 6.000 in.

Drilling method: Hammer drilled

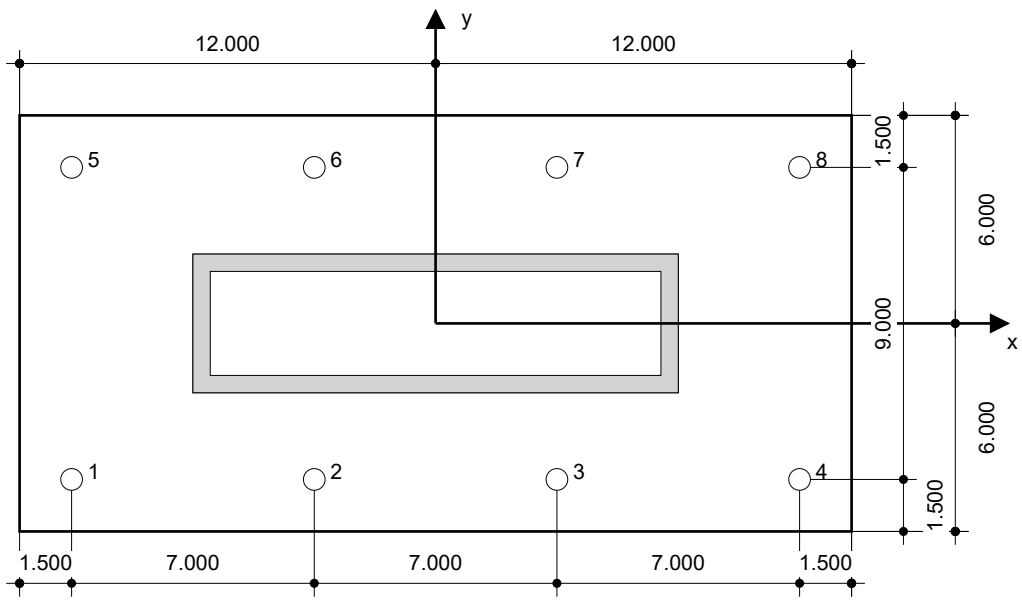
Minimum thickness of the base material: 7.250 in.

Cleaning: No cleaning of the drilled hole is required

Hilti ∅ 1/2 in KWIK-X (Carbon Steel) with KH-EZ with KHC 1/2" LARGE with 5 in nominal embedment depth per ICC-ES ESR-5065, Hammer drill bit installation per MPII

1.7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• No accessory required	• SIW 6-A22 Impact Screw Driver



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-10.500	-4.500	-	-	19.000	28.000
2	-3.500	-4.500	-	-	19.000	28.000
3	3.500	-4.500	-	-	19.000	28.000
4	10.500	-4.500	-	-	19.000	28.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	-10.500	4.500	-	-	28.000	19.000
6	-3.500	4.500	-	-	28.000	19.000
7	3.500	4.500	-	-	28.000	19.000
8	10.500	4.500	-	-	28.000	19.000

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

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 12.000 in x 0.625 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2, h_{ef} = 5.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.625 in
Profile:	HSS14X4X.500; (L x W x T x FT) = 14.000 in x 4.000 in x 0.500 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Uncracked concrete; 3000; $f_{c,cyl}$ = 3,000 psi; h = 8.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Welds [%]	Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]		
1	Combination 1	18,201	0.00	35,416	0.01	3	78	20

2.3 Anchor plate classification

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

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	0 lb	469 lb
Anchor 2	70 lb	0 lb
Anchor 3	1,280 lb	2,627 lb
Anchor 4	2,491 lb	2,751 lb
Anchor 5	69 lb	115 lb
Anchor 6	1,279 lb	2,470 lb
Anchor 7	2,490 lb	6,926 lb
Anchor 8	3,701 lb	3,900 lb

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

2.4 Profile/Stiffeners/Plate

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

2.4.1 Equivalent stress and plastic strain

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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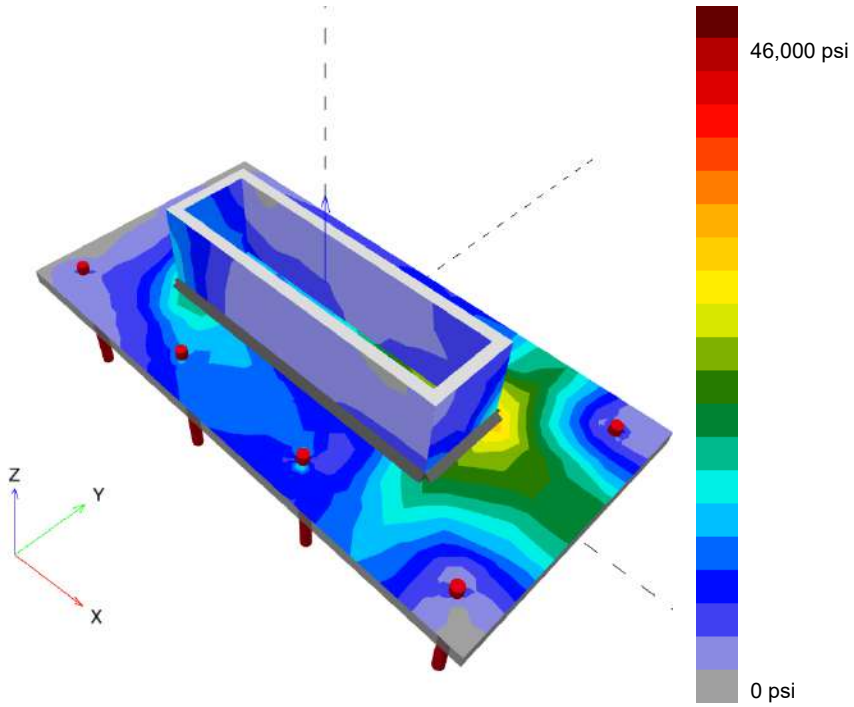
Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	35,416	0.01	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	18,201	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	11,263	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	10,732	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	8,346	0.00	OK

2.4.1.1 Equivalent stress

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



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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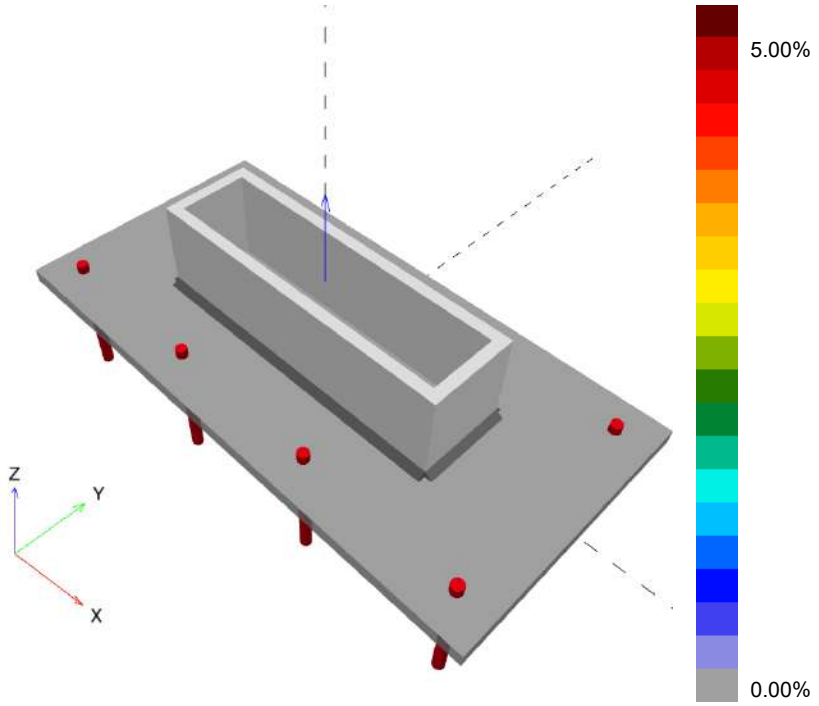
2.4.1.2 Plastic strain

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



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

Decisive load combination: 1 - Combination 1

Equations

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

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	1.225	0.625	58,000	0.500	43,514
Anchor 2	8.309	0.625	58,000	0.500	43,514
Anchor 3	6.375	0.625	58,000	0.500	43,514
Anchor 4	6.375	0.625	58,000	0.500	43,514
Anchor 5	1.220	0.625	58,000	0.500	43,514
Anchor 6	7.426	0.625	58,000	0.500	43,514
Anchor 7	9.277	0.625	58,000	0.500	43,514
Anchor 8	14.158	0.625	58,000	0.500	43,514

Results

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	731	32,635	3	OK
Anchor 2	767	32,635	3	OK
Anchor 3	799	32,635	3	OK
Anchor 4	758	32,635	3	OK
Anchor 5	719	32,635	3	OK
Anchor 6	750	32,635	3	OK
Anchor 7	748	32,635	3	OK
Anchor 8	682	32,635	3	OK

2.5 Welds

Profiles are modeled without taking the corner radius into account. Special rules for welding (e.g. for cold-formed profiles ...) are not taken into account by the software.

2.5.1 Anchor plate to profile

Decisive load combination: 1 - Combination 1

Equations

$F_{nw} = 0.6\, F_{EXX}\, (1.0 + 0.5\sin^{1.5}\Theta)$

$\Phi R_n = \Phi\, F_{nw}\, A_w$

$\text{Utilization} = \frac{F_n}{\Phi\, R_n}$

Variables

Edge	X_u	T_h [in]	L_s [in]	L [in]	L_c [in]	F_{EXX} [psi]	Θ [°]	A_w [in²]
Member 1-tfl 1	E70xx	◢0.220	0.311	3.994	1.331	70,000	74.7	0.29
Member 1-bfl 1	E70xx	0.220◣	0.311	3.994	1.331	70,000	74.5	0.29
Member 1-w 1	E70xx	◢0.220	0.311	12.982	1.442	70,000	78.8	0.32
Member 1-w 2	E70xx	0.220◣	0.311	12.982	1.442	70,000	85.5	0.32

Results

Edge	F_n [lb]	ΦR_n [lb]	Utilization [%]	Status
Member 1-tfl 1	10,494	13,598	78	OK
Member 1-bfl 1	10,437	13,589	77	OK
Member 1-w 1	11,250	14,851	76	OK
Member 1-w 2	11,274	14,971	76	OK

2.6 Concrete

Decisive load combination: 1 - Combination 1

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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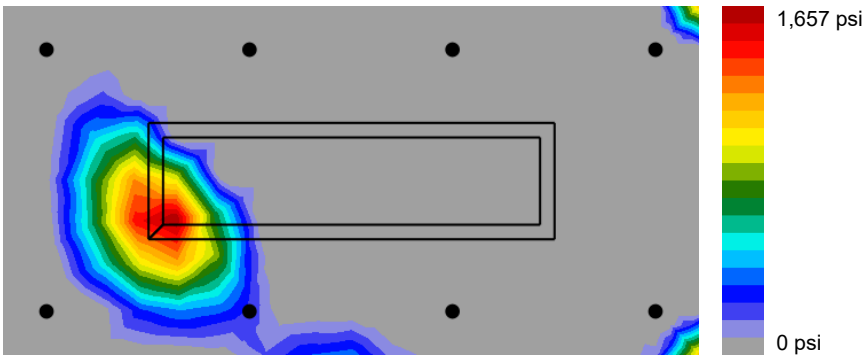
2.6.1 Compression in concrete under the anchor plate



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

Equations

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

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

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

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

Variables

N [lb]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
31,425	3,000	0.65	47.65	1,429.28

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,315	659	20	OK

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

A_1	Loaded area of concrete
A_2	Supporting area
A_w	Effective area of weld critical element
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_{EXX}	Electrode classification number, i.e. minimum specified tensile strength
F_u	Specified minimum tensile strength of the connected material
F_n	Force in weld critical element
F_{nw}	Nominal stress of the weld material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
L	Length of weld
L_c	Length of weld critical element
L_s	Leg size of weld
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
R_n	Resistance
t	Thickness of the anchor plate
Θ	Angle of loading measured from the weld longitudinal axis
T_h	Throat thickness of weld
V	Resultant of shear forces Vy, Vz in bolt.
X_u	Filler metal tensile strength

2.8 Warnings

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

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

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

	Load combination	Max. utilization	Status
Anchors	Combination 1	99%	OK
Anchor plate	Combination 1	99%	OK
Welds	Combination 1	78%	OK
Concrete	Combination 1	20%	OK
Profile	Combination 1	40%	OK

Fastening meets the design criteria!

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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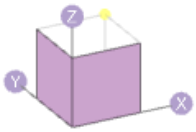
LANDING STRINGER CONNECTION TO CMU (NODE N9)

LOADS (RISA REACTIONS):

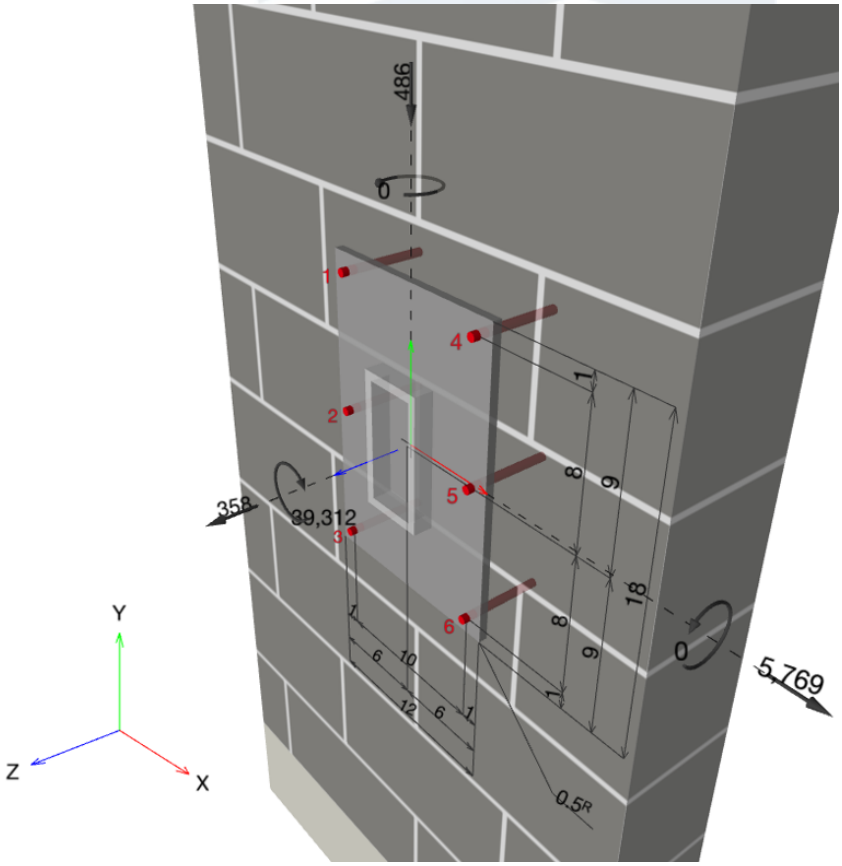
ASD:
Rx= 5769 lb
Ry= 358 lb
Rz= 486 lb
My= 39312 lb-in



LOADS ASD TO HILTI:

ASD HILTI:
Rx= 5769 lb ("Rx" HILTI)
Ry= 358 lb ("Rz" HILTI)
Rz= 486 lb ("Ry" HILTI)
My= 39312 lb-in ("Mz" HILTI)

* For anchor design see spreadsheet printout



CONN. SKETCH

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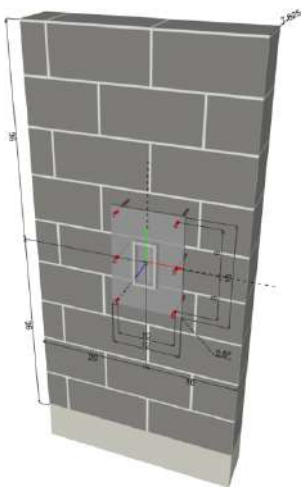
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 1/2 (4 1/4)**
Item number:	418076 KH-EZ 1/2"x5"
Specification text:	Hilti ∅ 1/2 (4 1/4) KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 4.25 in nominal embedment depth per Technical data , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 4.250 in.
Material:	Carbon Steel
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 18.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS8X4X.500; (L x W x T) = 8.000 in. x 4.000 in. x 0.500 in.
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

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

Geometry [in.]



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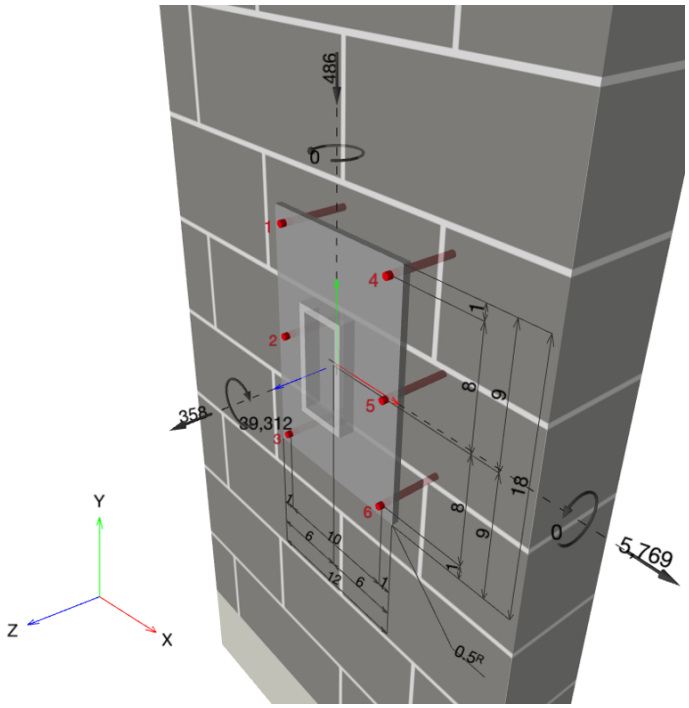
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 358; V_x = 5,769; V_y = -486; M_x = 0; M_y = 0; M_z = -39,312;	no	96

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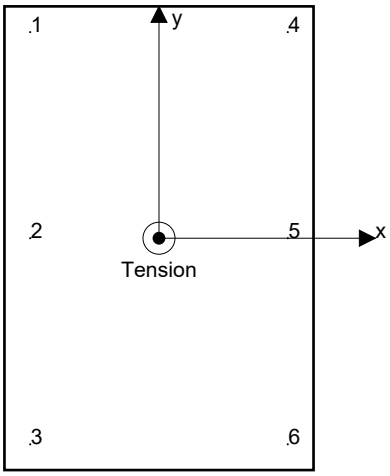
2 Load case/Resulting anchor forces



Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	60	1,782	1,736	403
2	60	1,043	961	403
3	60	444	187	403
4	60	1,826	1,736	-565
5	60	1,115	961	-565
6	60	595	187	-565

max. compressive strain: - [‰]
max. compressive stress: - [psi]
Resulting tension force in (x/y)=(-0.000/-0.000): 358 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3 Tension load (Most utilized anchor 4)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Overall strength	60	1,212	5	OK

3.1 Overall strength

$P_{t,Base}$ = Value refer to Hilti Technical Data

$P_t = P_{t,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp} \cdot f_{red,Bedjoint}$

$P_t \geq P_s$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	8.000	4.000	8.000	68

Results

P_t [lb]	$P_{t,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$	$f_{red,Bedjoint}$
1,212	1,212	60	1.000	1.000	1.000	1.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load (Most utilized anchor 4)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Overall strength para and perp, (Dir. x+)[1]	-	-	95	OK

[1]Shear utilization may result from parallel and perpendicular shear (see details)

4.1 Overall strength parallel

$V_{t,Base,||}$ = Value refer to Hilti Technical Data
$V_{t,||} = V_{t,Base,||} \cdot f_{red,E,||} \cdot f_{red,s,||} \cdot f_{red,Temp}$
$V_{t,||} \geq V_{s,||}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	8.000	4.000	8.000	68

Results

$V_{t,||}$ [lb]	$V_{t,Base,||}$ [lb]	$V_{s,||}$ [lb]	$f_{red,E,||}$	$f_{red,S,||}$	$f_{red,Temp}$	Utilization $\beta_{V,||}$ [%]
1,959	1,959	-565	1.000	1.000	1.000	29

4.2 Overall strength perpendicular

$V_{t,Base,\perp}$ = Value refer to Hilti Technical Data
$V_{t,\perp} = V_{t,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$
$V_{t,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	8.000	4.000	8.000	68

Results

$V_{t,\perp}$ [lb]	$V_{t,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
1,959	1,959	1,736	1.000	1.000	1.000	89

4.3 Shear interaction

$\beta_{V,||} = \frac{V_{s,||}}{V_{t,||}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.288	0.886	1.667	95	OK

$\beta_V = \beta_{V,||}^{\delta} + \beta_{V,\perp}^{\delta} <= 1.0$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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5 Combined tension and shear loads (Most utilized anchor 4)

$\beta_P = \frac{P_s}{P_t}$	$\beta_{V,\|\|} = \frac{V_{s,\|\|}}{V_{t,\|\|}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	α	Utilization $\beta_{P,V}$ [%]	Status
0.007	0.288	0.818	1.667	96	OK

$\beta_{P,V} = \beta_P^{\alpha} + \beta_{V,\|\|}^{\alpha} + \beta_{V,\perp}^{\alpha} <= 1.0$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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7 Installation data

Profile: Rectangular HSS (AISC), HSS8X4X.500; (L x W x T) = 8.000 in. x 4.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = - in.

Plate thickness (input): 0.500 in.

Drilling method: Drilled in hammer mode

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) (Carbon Steel) 1/2 (4 1/4)

Item number: 418076 KH-EZ 1/2"x5"

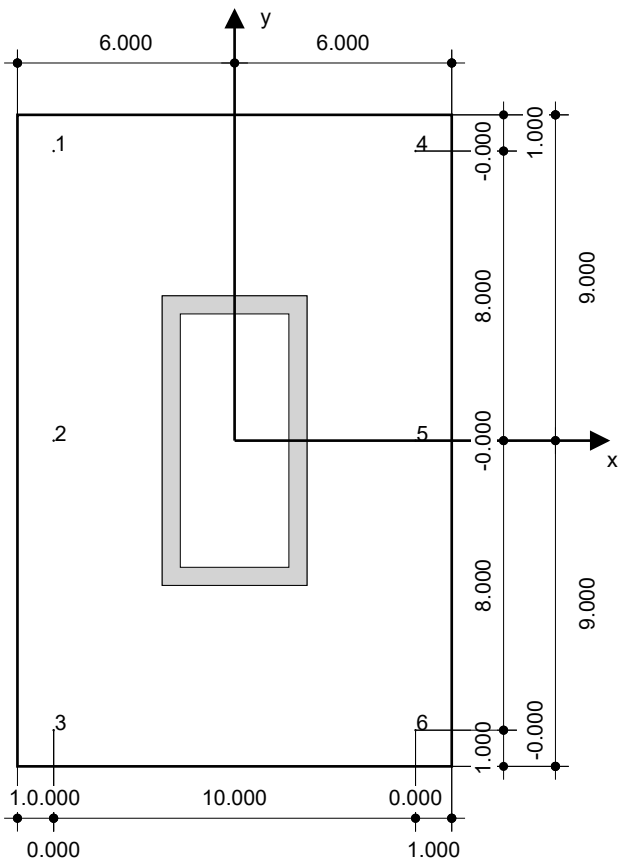
Maximum installation torque: 408 in.lb

Hole diameter in the base material: 0.500 in.

Hole depth in the base material: 4.625 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 1/2 (4 1/4) KWIK HUS-EZ (KH-EZ) (Carbon Steel) with 4.25 in nominal embedment depth per Technical data , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.000	8.000	15.000	21.000	44.000	28.000
2	-5.000	-0.000	15.000	21.000	36.000	36.000
3	-5.000	-8.000	15.000	21.000	28.000	44.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	5.000	8.000	25.000	11.000	44.000	28.000
5	5.000	-0.000	25.000	11.000	36.000	36.000
6	5.000	-8.000	25.000	11.000	28.000	44.000

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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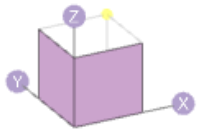
TOP CONNECTION DESIGN (NODE N2)

LOADS (RISA REACTIONS):

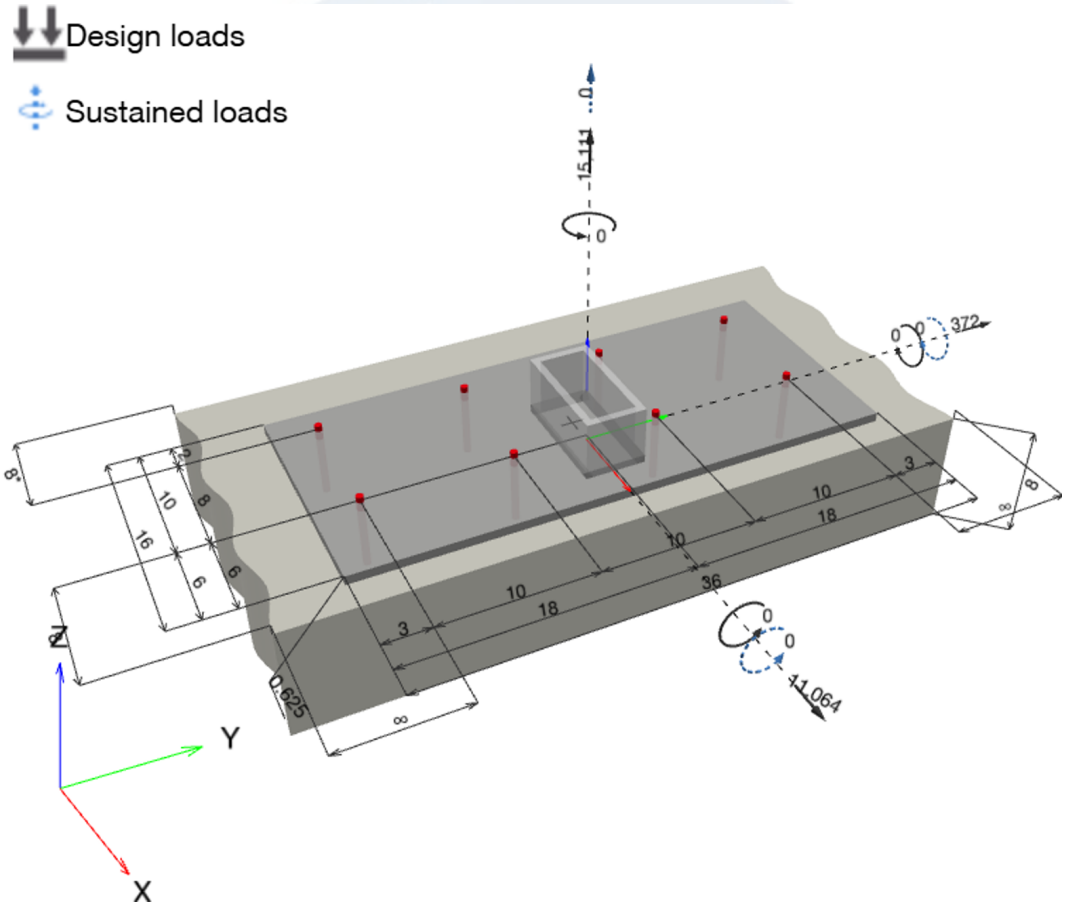
ASD:
Rx= 9444 lb
Ry= 232 lb
Rz= 6915 lb



LOADS PROCESSING (ASD TO LRFD):

LRFD:
Rx= 9444 lb x 1.60 = 15111 lb ("Rz" HILTI)
Ry= 232 lb x 1.60 = 372 lb ("Ry" HILTI)
Rz= 6915 lb lb x 1.60 = 11064 lb ("Rx" HILTI)

* For anchor design see spreadsheet printout



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Specifier's comments:

1 Anchor Design

1.1 Input data

Anchor type and diameter:	**KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2**
Item number:	418076 KH-EZ 1/2"x5" (element) / 2362252 KHC 1/2" LARGE (capsule)
Specification text:	Hilti ∅ 1/2 in KWIK-X (Carbon Steel) with KH-EZ with KHC 1/2" LARGE with 4.25 in nominal embedment depth per ICC-ES ESR-5065, Hammer drill bit installation per MPII,
Effective embedment depth:	$h_{ef,act}$ = 4.250 in. ($h_{ef,limit}$ = - in.), h_{nom} = 4.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-5065
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-19 / Chem
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.625 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 16.000 in. x 36.000 in. x 0.625 in.;
Profile:	Rectangular HSS (AISC), HSS8X4X.500; (L x W x T) = 8.000 in. x 4.000 in. x 0.500 in.
Base material:	uncracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in., Temp. short/long: 32/32 °F
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

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

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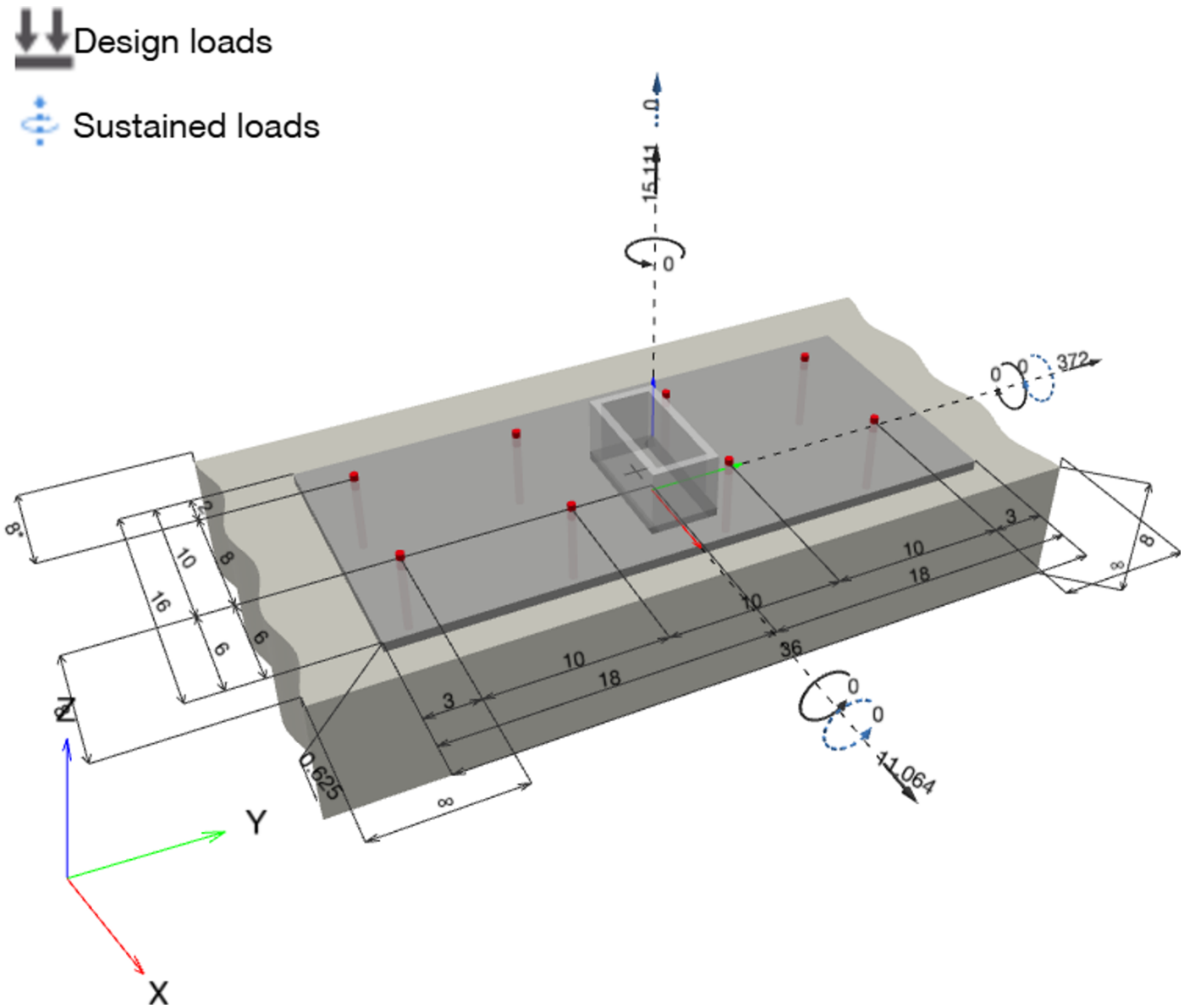
Geometry [in.] & Loading [lb, in.lb]



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1.1.1 Design results

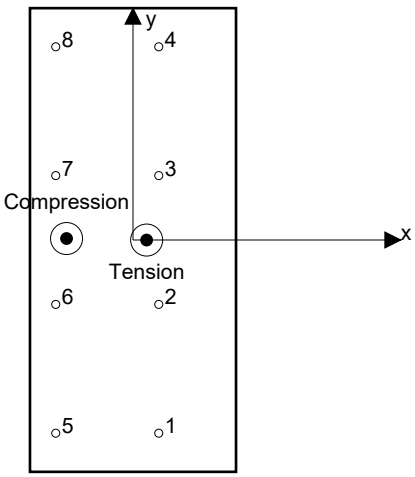
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 15,111; V_x = 11,064; V_y = 372; M_x = 0; M_y = 0; M_z = 0;	no	90

1.2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	691	1,333	1,332	53
2	7,108	1,482	1,480	75
3	7,255	1,467	1,467	28
4	623	1,293	1,292	51
5	8	1,324	1,319	116
6	1,025	1,457	1,454	88
7	1,066	1,442	1,442	-5
8	3	1,280	1,280	-34



Resulting tension force in (x/y)=(1.059/-0.005): 17,780 [lb]
Resulting compression force in (x/y)=(-5.160/0.114): 5,243 [lb]

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

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	7,255	11,777	62	OK
Bond Strength**	17,780	28,670	63	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	17,780	27,092	66	OK

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

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

N_{sa} = ESR value refer to ICC-ES ESR-5065
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

N_{sa} [lb]
18,119

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,119	0.650	11,777	7,255

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

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na} \psi_{ec2,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba}$ ACI 318-19 Eq. (17.6.5.1b)

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

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2 c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,313	0.500	4.250	8.000	1.000	2,313

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
3.054	0.009	9.404	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.218	996.99	208.41	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
0.703	0.999	0.851	15,444

Results

N_{ag} [lb]	ϕ_{bond}	ϕN_{ag} [lb]	N_{ua} [lb]
44,108	0.650	28,670	17,780

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

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

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.250	3.054	0.009	8.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.404	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
887.07	162.57	0.676	0.999	1.000	0.851	13,299

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
41,681	0.650	27,092	17,780

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

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,482	5,547	27	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	11,070	84,582	14	OK
Concrete edge failure in direction x+**	11,072	19,140	58	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-5065
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

V_{sa} [lb]
9,245

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
9,245	0.600	5,547	1,482

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.2 Pryout Strength (Concrete Breakout Strength controls)

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

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

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

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

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

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.250	0.005	0.134	8.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	9.404	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
887.07	162.57	0.999	0.979	1.000	0.851	13,299

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
120,832	0.700	84,582	11,070

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.3 Concrete edge failure in direction x+

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

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

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

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

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

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

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

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
8.000	-	0.119	1.400	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
4.000	1.000	0.500	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
432.00	288.00	0.990	1.000	1.225	10,737

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
27,343	0.700	19,140	11,072

*Anchor row defined by: Anchor 1-4; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.656	0.578	5/3	90	OK

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

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

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.7 Installation data

Anchor type and diameter: KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2

Profile: Rectangular HSS (AISC), HSS8X4X.500; (L x W x T) = 8.000 in. x 4.000 in. x 0.500 in.

Item number: 418076 KH-EZ 1/2"x5" (element) / 2362252 KHC 1/2" LARGE (capsule)

Hole diameter in the fixture: d_f = 0.625 in.

Maximum installation torque: -

Plate thickness (input): 0.625 in.

Hole diameter in the base material: 0.500 in.

Hole depth in the base material: 5.250 in.

Drilling method: Hammer drilled

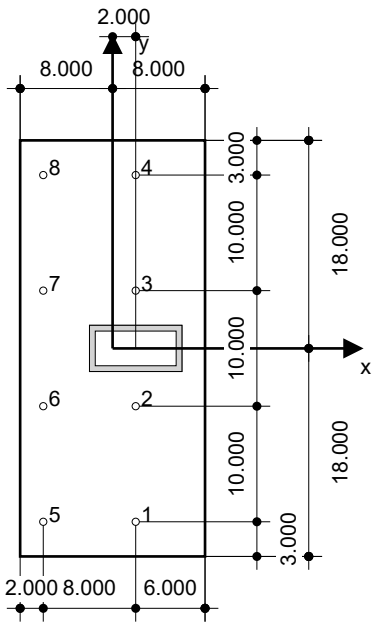
Minimum thickness of the base material: 6.500 in.

Cleaning: No cleaning of the drilled hole is required

Hilti ∅ 1/2 in KWIK-X (Carbon Steel) with KH-EZ with KHC 1/2" LARGE with 4.250039 in nominal embedment depth per ICC-ES ESR-5065, Hammer drill bit installation per MPII

1.7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• No accessory required	• SIW 6-A22 Impact Screw Driver



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	2.000	-15.000	16.000	8.000	-	-
2	2.000	-5.000	16.000	8.000	-	-
3	2.000	5.000	16.000	8.000	-	-
4	2.000	15.000	16.000	8.000	-	-

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	-6.000	-15.000	8.000	16.000	-	-
6	-6.000	-5.000	8.000	16.000	-	-
7	-6.000	5.000	8.000	16.000	-	-
8	-6.000	15.000	8.000	16.000	-	-

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

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 16.000 in x 36.000 in x 0.625 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	KWIK-X (Carbon Steel) 1/2 (4 1/4) hnom2, h_{ef} = 4.250 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.625 in
Profile:	HSS8X4X.500; (L x W x T x FT) = 8.000 in x 4.000 in x 0.500 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 2.000 in Eccentricity y: 0.000 in
Base material:	Uncracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 8.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	19,843	0.00	26,786	0.00	5	3

2.3 Anchor plate classification

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

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	3,678 lb	691 lb
Anchor 2	3,680 lb	7,108 lb
Anchor 3	3,681 lb	7,255 lb
Anchor 4	3,681 lb	623 lb
Anchor 5	547 lb	8 lb
Anchor 6	549 lb	1,025 lb
Anchor 7	550 lb	1,066 lb
Anchor 8	550 lb	3 lb

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

2.4 Profile/Stiffeners/Plate

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

2.4.1 Equivalent stress and plastic strain

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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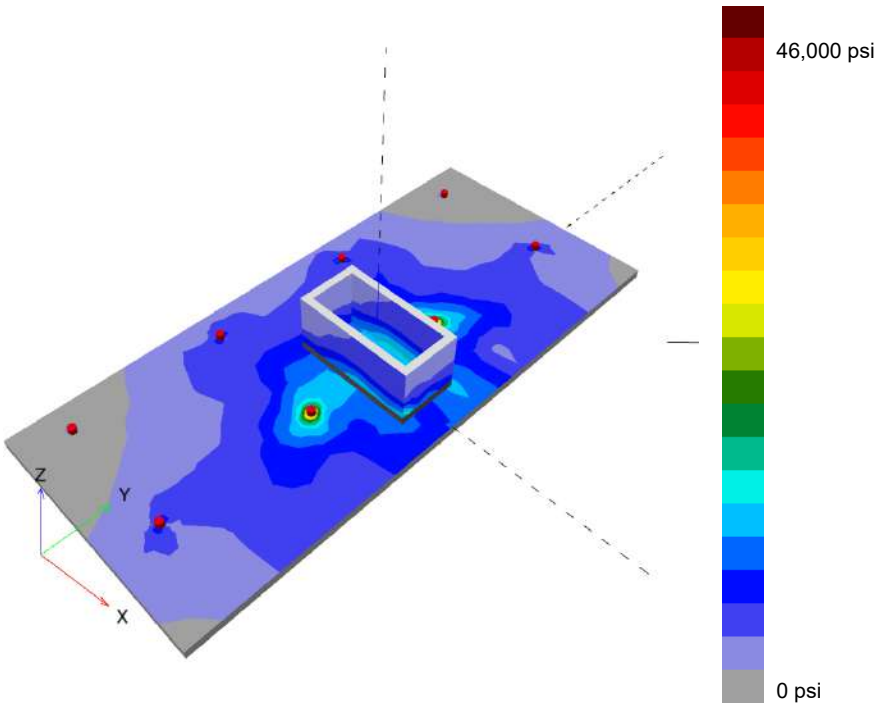
Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	26,786	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	13,638	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	19,843	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	14,783	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	15,407	0.00	OK

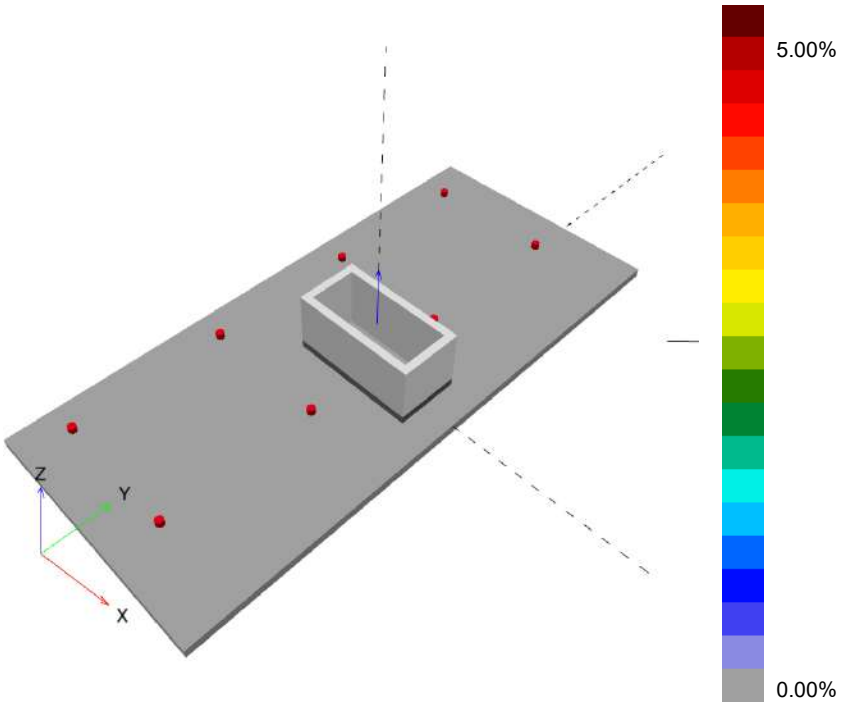
2.4.1.1 Equivalent stress

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



2.4.1.2 Plastic strain

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



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

Decisive load combination: 1 - Combination 1

Equations

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

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	7.375	0.625	58,000	0.500	43,514
Anchor 2	7.375	0.625	58,000	0.500	43,514
Anchor 3	7.375	0.625	58,000	0.500	43,514
Anchor 4	7.375	0.625	58,000	0.500	43,514
Anchor 5	1.695	0.625	58,000	0.500	43,514
Anchor 6	1.691	0.625	58,000	0.500	43,514
Anchor 7	1.688	0.625	58,000	0.500	43,514
Anchor 8	1.688	0.625	58,000	0.500	43,514

Results

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	TOP CONNECTION_V01	Date:	7/18/2025
Fastening point:			

	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	1,333	32,635	5	OK
Anchor 2	1,482	32,635	5	OK
Anchor 3	1,467	32,635	5	OK
Anchor 4	1,293	32,635	4	OK
Anchor 5	1,324	32,635	5	OK
Anchor 6	1,457	32,635	5	OK
Anchor 7	1,441	32,635	5	OK
Anchor 8	1,280	32,635	4	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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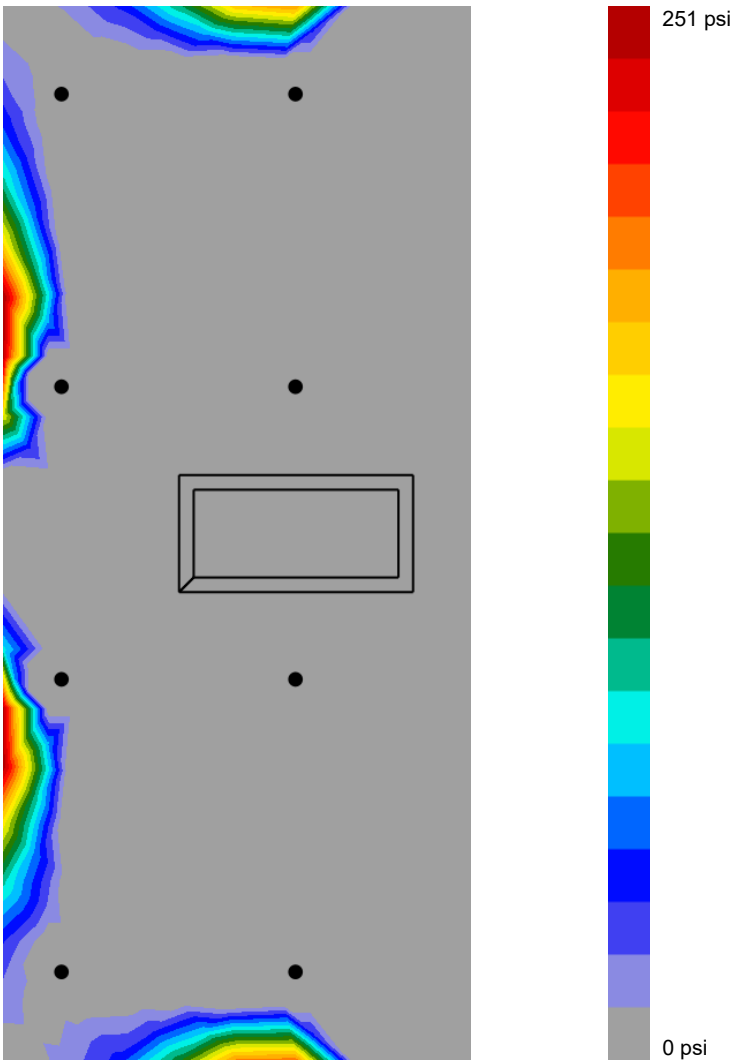
Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	TOP CONNECTION_V01	Date:	7/18/2025
Fastening point:			

2.5.1 Compression in concrete under the anchor plate



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

Equations

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

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

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

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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	TOP CONNECTION_V01	Date:	7/18/2025
Fastening point:			

Variables

N [lb]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
5,243	4,000	0.65	54.55	867.31

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,420	96	3	OK

2.6 Symbol explanation

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

2.7 Warnings

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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	TOP CONNECTION_V01	Date:	7/18/2025
Fastening point:			

3 Summary of results

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

	Load combination	Max. utilization	Status
Anchors	Combination 1	90%	OK
Anchor plate	Combination 1	75%	OK
Concrete	Combination 1	3%	OK
Profile	Combination 1	44%	OK

Fastening meets the design criteria!

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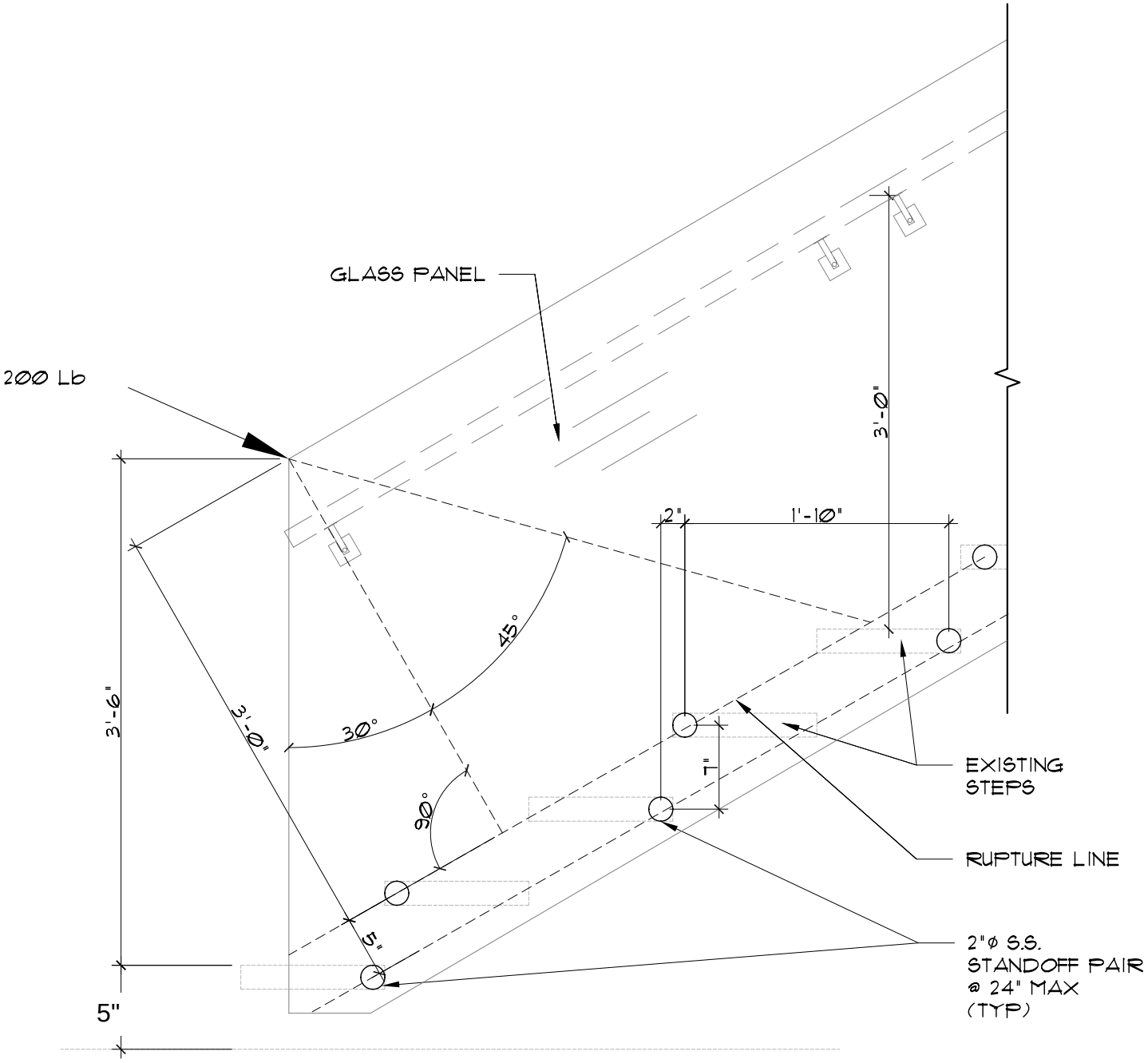

**RIGID PANEL DISTRIBUTES LOAD EVENLY BETWEEN BOTH STANDOFF PAIR

CONSERVATIVE ANALYSIS

Min 2 Standoff pairs by glass panel

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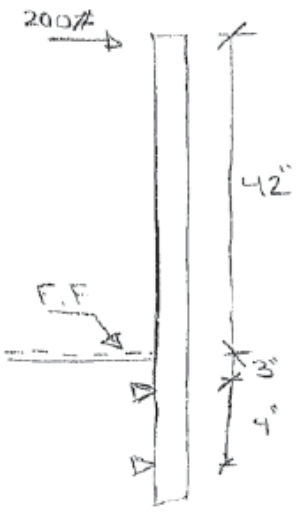
Min Glass Width



(Fb) Glass allowable Bending = $\frac{\text{Modulus of rupture of glass in flexure}}{\text{Safety Factor}}$

$Fb_{glass} = 24000/4$

$Fb_{glass} = 6000$ psi.

$S_x = \frac{(\text{glass thk})^2 \times \text{min glass width}(W_{min})}{6}$

$h_{max} = 42" + 3" = 45"$

glass thk = $9/16"$

$M_{max} = 200 \times h_{max}$

$M_{max} = 200 \times 45$

$M_{max} = 9000$ #-in.

$fb = \frac{M_{max}}{S_x} \leq Fb = 6000$ psi

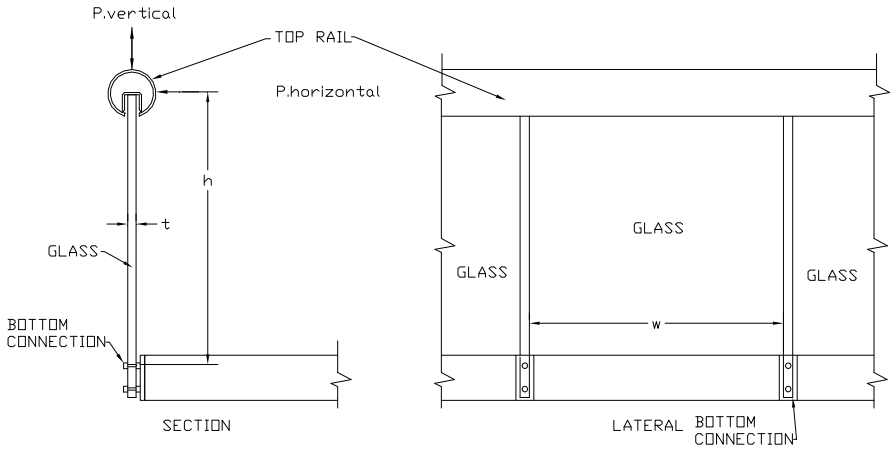
$S_x = \frac{M_{max}}{fb}$

$\frac{(\text{glass thk})^2 \times W_{min}}{6} = \frac{M_{max}}{Fb}$

$\therefore W_{min} = \frac{6\, M_{max}}{Fb \times (\text{glass thk})^2} = \frac{6 \times 9000\#\text{-in}}{6000\text{psi} \times (9/16)^2} = 28.4"$

$W_{min} = 29"$

Glass Railing Design w/ Standoff Support



Loads Data:

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

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

$q_{wind} := 5.0$ 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 (psi)

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

$SF := 4.00$ Safety Factor

Geometric Glass Railing Data:

$h := 45.00$ Height of Glass Lite Pannel in Cantilever (in)

$t := 0.5625$ Thickness of Glass Pannel (in)

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

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

Then

$F_b := \frac{Mr_{flexure}}{SF}$ $F_b = 6000.00$ 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

$S_x := \frac{L \cdot t^2}{6}$ $S_x = 1.58$ in^3

$I_x := \frac{L \cdot t^3}{12}$ $I_x = 0.44$ in^4

$A := L \cdot t$ $A = 16.88$ in^2

Actual Glass Moment:

Concentrated Load = 200 lbs.

$M_{200} := P_{200} \cdot h$ $M_{200} = 9000.00$ lbs – in

Uniform Load = 50 plf

$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$ $M_{50} = 5625.00$ lbs – in

Uniform Distributed Wind Load

$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$ $M_{wind} = 1054.69$ lbs – in

$M_{actual} := \max(M_{200}, M_{50}, M_{wind})$ $M_{actual} = 9000.00$ lbs – in

Actual Glass Shear:

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

$$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$$ $V_{wind} = 46.88$ lbs

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

Section Required:

Bending Design:

Section Modulus Required

$$Sx_r := \frac{M_{actual}}{F_b}$$ $Sx_r = 1.5$ in^3

Shear Design:

Area Required

$$A_r := \frac{V_{actual}}{F_v}$$ $A_r = 0.07$ 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 Deflection:

$$\Delta_{allow} := \frac{h}{30}$$ $\Delta_{allow} = 1.50$ in

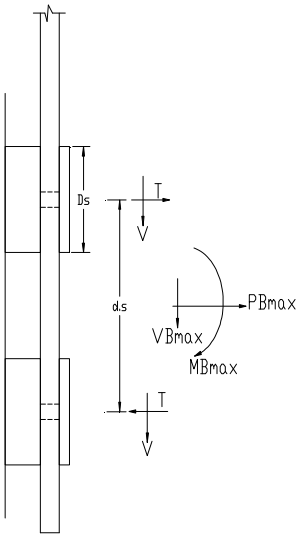
$$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_x}$$ $\Delta_{200} = 1.31$ in

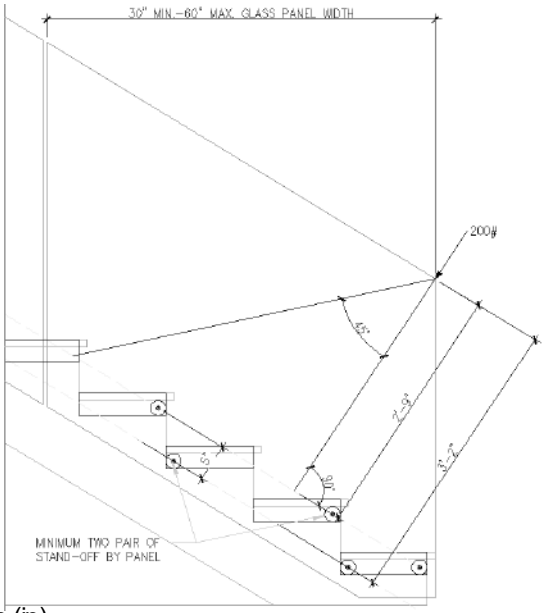
$$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_x}$$ $\Delta_{wind} = 0.12$ in

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

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)

$P_{system} := 50.00$ Pannel Total Weigth of Glass and Handrail (lbs)

Maximum Connection Moment:

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

$M_{Bactual} = 4500.00$ lbs – in

Maximum Connection Shear:

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

$V_{Bactual} = 125.00$ lbs

Maximum Connection Axial:

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

$P_{Bactual} = 100.00$ lbs

Tensile Load Per Glass Supports in Connection:

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

$T = 1225.00$ lbs

Shear Load Per Glass Supports in Connection:

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

$V = 125.00$ lbs

Check Two Way Glass Shear at Glass Supports in Connection:

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

$A_p = 8.05$ in^2

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

$f_v = 270.52$ psi

$$SHEAR_{twoway} := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq F_v \end{cases}$$

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

S.S Bolt Tension & Shear Stress Design 15th

Data:

$T_{bolt} := 1.225$ Tension in Anchor (kips)

$\Omega := 2.25$ ASD Factor=2.00

$V_{bolt} := 0.125$ Shear in Anchor (kips)

$F_{nv} := 27.00$ Nominal Shear Strength (ksi) Table J3.2 AISC 16.1-120 .....=0.563*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)

$n := 16.00$ Threads per in..... (AISC-Table 7-17 page 7-81)

$$F_v := \frac{F_{nv}}{\Omega}$$

$F_v = 12.00$ ksi

$$F_t := \frac{F_{nt}}{\Omega}$$

$F_t = 20.00$ ksi

$$A_{tension} := 0.7854 \cdot \left(D - \frac{0.9743}{n} \right)^2$$

$A_{tension} = 0.08$ in^2

$$A_{shear} := \pi \cdot \left(\frac{D}{2} \right)^2$$

$A_{shear} = 0.11$ in^2

Check Shear

$$f_v := \frac{V_{bolt}}{A_{shear}}$$

$f_v = 1.13$ ksi

$$V_{allow} := F_v \cdot A_{shear}$$

$V_{allow} = 1.33$ kips

$$SHEAR := \text{if}\left(f_v \le F_v, \text{"OK"}, \text{"NG"} \right)$$

$SHEAR = \text{"OK"}$

Check Tension

$$f_t := \frac{T_{bolt}}{A_{tension}}$$

$f_t = 15.81$ ksi

$$F'_{nt} := \min\left(F_t, \max\left(0, 1.3F_t - \Omega \cdot \frac{F_t}{F_v} \cdot f_v \right) \right)$$

$F'_{nt} = 20.00$ ksi

$$T_{allow} := F'_{nt} \cdot A_{tension}$$

$T_{allow} = 1.55$ kips

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

$TENSION = \text{"OK"}$

Minimum Thread Length

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

$TL_{min} = 0.21$ in

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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=0.110 (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 14th

Load Data:

$T_{bolt} := 1.225$ Tension in Anchor (kips)

Bolt Data:

$D := 0.375$ Bolt Diameter (in)

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

Base Material Data:

$Fv_{base} := 14.40$ 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

$Fu_{table} := 60.00$ Maximun 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)

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

$TL_{calc} = 0.172$ in

Minimum Thread Length (by Equivalent AISC Standard)

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

$TL_{equivalent} = 0.128$ in

Minimum Thread Length

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

$TL_{min} = 0.172$ in

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

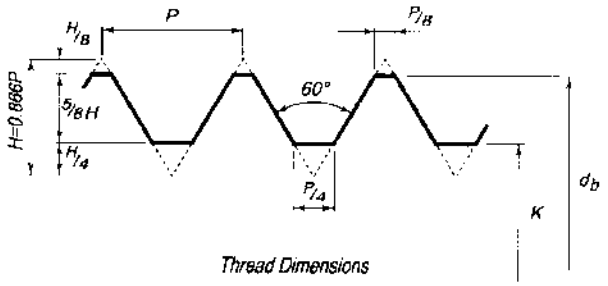
ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

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

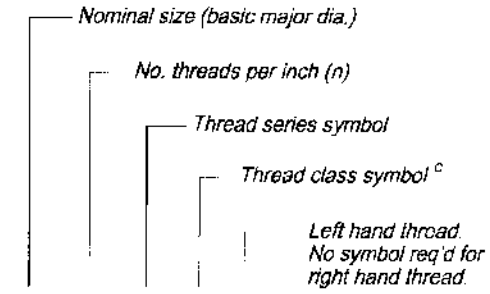
SCREW THREADS

Unified Standard Series-UNC/UNRC and 4UN/4UNR

ANSI B1.1



Thread Dimensions



Standard Designations

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 $= 0.7854 \times \left(d_b - \frac{0.9743}{n}\right)^2$

[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			■							▨	
	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							▨		▨	▨	▨
	Gr. 50	50	65	to 4							▨		▨	▨	▨
	Gr. 55	55	70	to 2							▨		▨	▨	▨
	Gr. 60	60	75	to 1.25							▨		▨	▨	▨
	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									■	■	■
	Gr. 55	55	75-95	0.25 to 4									▨	▨	▨
	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* ≤ 6 in.	*L* > 6 in.
Bolts	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.

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PROJ. NAME: _____

PROJ No.: _____ DATE: _____

DESIGNED BY: _____ PAGE: _____

REVIEWED BY: _____

CONNECTION TO CONCRETE BEAM.

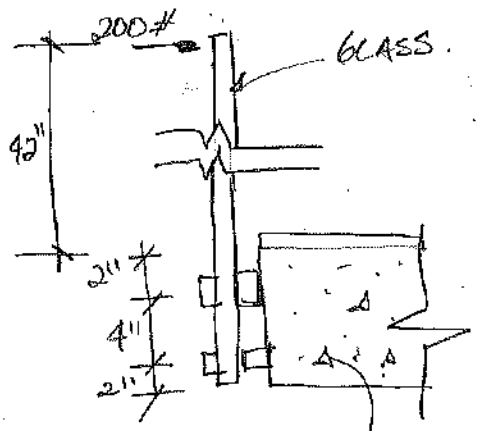
Load

$P = 200\#$

Load processing

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

$T = 2400\ \#$



Concrete beam at 2nd Floor

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

$$T_{bolt} = \frac{2400\#}{2\ bolt} = 1200\ \#$$

$$V \text{ per bolt} = \frac{200\#}{2} = 100\ \#$$

* Use Ø 3/8" Threaded Rod C&T Every Epoxy w/3-3/8" min embedment, 4" min spacing & 2" min edge dist.

$T_{allow} = 1858\ \#\ > 1200\ \#$ (OK.)

$V_{allow} = 405\ \#\ > 100\ \#$ (OK.)

COMBINED

1200# / 1853# + 100# / 405# < 1.00
0.90 < 1.00 (OK)

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C6 +
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Concrete		3000
Diameter		Ø 3/8
Embed		3 3/8
Steel Strength		ASTM A307
Spacing=		4.00
Edge Dist.=		2.00
Double Edge Dist		No
by Design	Tbolt =	1200.00
	Vbolt =	100.00
by Manuf.	T =	2080.00
	V =	1040.00

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	→	0.75 x Anchor Embedment
Minimum Edge Distance—Tension		
70% Tension Load	→	0.25 x Anchor Embedment
Critical Edge Distance—Shear		
100% Shear Load	→	1.25 x Anchor Embedment
Minimum Edge Distance—Shear		
10% Shear Load	→	0.25 x Anchor Embedment

LOAD FACTOR		DISTANCE FROM ANOTHER ANCHOR
Critical Spacing—Tension		
100% Tension Load	→	1.25 x Anchor Embedment
Minimum Spacing—Tension		
80% Tension Load	→	0.25 x Anchor Embedment
Critical Spacing—Shear		
100% Shear Load	→	1.25 x Anchor Embedment
Minimum Spacing—Shear		
25% Shear Load	→	0.25 x Anchor Embedment

Find interpolated value of 'B' for the given value of 'A' from table of data below:

Tension

	Edge Dist.	2
0.75 x Embed	2.53125	1
	2	**0.91**
0.25 x Embed	0.84375	0.7

Tension

	Spacing=	4
1.25 x Embed	4.21875	1
	4	**0.98**
0.25 x Embed	0.84375	0.75

Shear

	Edge Dist.	2
1.25 x Embed	4.21875	1
	2	**0.41**
0.25 x Embed	0.84375	0.1

Shear

	Spacing=	4
1.25 x Embed	4.21875	1
	4	**0.95**
0.25 x Embed	0.84375	0.3

Tallow & Vallow	Tallow=	1853.00	>	Tbolt	**O.K.**
	Vallow=	405.00	>	Vbolt	**O.K.**

Combine	$= (\frac{T_{bolt}}{T_{allow}})^{\frac{5}{3}} + (\frac{V_{bolt}}{V_{allow}})^{\frac{5}{3}} =$	0.58	<	1.00	**O.K.**

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Threaded Rod Allowable Tension Loads Installed in Solid Concrete

THREADED ROD DIA. (In.)	MIN. EMBEDMENT DEPTH (In.)	ALLOWABLE TENSION LOAD BASED ON CONCRETE STRENGTH			ALLOWABLE TENSION LOAD BASED ON STEEL STRENGTH		
		3000	**5000**	**7000**	**ASTM A307**	**ASTM A193GR**	**ASTM F593**
Ø 3/8	1 1/2	790.0	945.0	1100.0	**2080.0**	4340.0	3995.0
	3 3/8	**2910.0**	3080.0	3245.0	**2080.0**	4340.0	3995.0
Ø 1/2	2	1520.0	1755.0	1990.0	3730.0	7780.0	7155.0
	4 1/2	5000.0	5825.0	6650.0	3730.0	7780.0	7155.0
Ø 5/8	2 1/2	2145.0	2500.0	2855.0	5870.0	12230.0	11250.0
	5 5/8	6225.0	7255.0	8280.0	5870.0	12230.0	11250.0
Ø 3/4	3	3010.0	3395.0	3775.0	8490.0	17690.0	14860.0
	6 3/4	9160.0	10675.0	12185.0	8490.0	17690.0	14860.0
Ø 7/8	3 1/2	3750.0	4335.0	4915.0	11600.0	25510.0	20835.0
	7 7/8	13895.0	17585.0	21275.0	11600.0	25510.0	20835.0
Ø 1	4	4435.0	5100.0	5760.0	15180.0	31620.0	26560.0
	9	15565.0	18465.0	21365.0	15180.0	31620.0	26560.0
Ø 1 1/4	5	8675.0	9235.0	9795.0	23800.0	49580.0	34670.0
	11 1/4	19455.0	22665.0	25875.0	23800.0	49580.0	34670.0

C6 +
For the Most Demanding Jobs

Threaded Rod Allowable Shear Loads Installed in Solid Concrete

THREADED ROD DIA. (In.)	MIN. EMBEDMENT DEPTH (In.)	ALLOWABLE TENSION LOAD BASED ON CONCRETE STRENGTH	ALLOWABLE TENSION LOAD BASED ON STEEL STRENGTH		
		3000	**ASTM A307**	**ASTM A193GR**	**ASTM F593**
Ø 3/8	1 1/2	N/A	**1040.0**	2170.0	1995.0
	3 3/8	**1300**	**1040.0**	2170.0	1995.0
Ø 1/2	2	N/A	1870.0	3895.0	3585.0
	4 1/2	2855	1870.0	3895.0	3585.0
Ø 5/8	2 1/2	N/A	2940.0	6125.0	5635.0
	5 5/8	4575	2940.0	6125.0	5635.0
Ø 3/4	3	N/A	4250.0	8855.0	7440.0
	6 3/4	6430	4250.0	8855.0	7440.0
Ø 7/8	3 1/2	N/A	5800.0	12760.0	10730.0
	7 7/8	8030	5800.0	12760.0	10730.0
Ø 1	4	N/A	7590.0	15810.0	13285.0
	9	9630	7590.0	15810.0	13285.0
Ø 1 1/4	5	N/A	11900.0	24790.0	18840.0
	11 1/4	16270	11900.0	24790.0	18840.0

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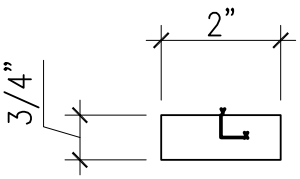
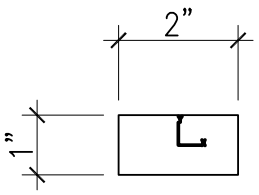
HANDRAIL AT WALL DESIGN

SECTION PROPERTIES



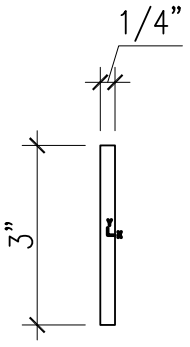
BRACKET 2"x ¾"
STEEL FLAT BAR, FY=36KSI

Area = 1.5000 in^2
Ixx = 0.0703 in^4
Iyy = 0.5000 in^4
Jz = 0.5703 in^4
Pxy = −0.0000 in^4
rxx = 0.2165 in
ryy = 0.5774 in
Sxx max = 0.1875 in^3
Sxx min = 0.1875 in^3
Syy max = 0.5000 in^3
Syy min = 0.5000 in^3

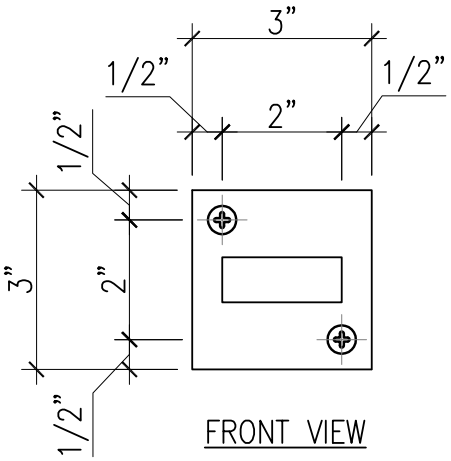


TOP RAIL (HANDRAIL) 2"x 1"
IPE WOOD, FB=5600PSI

Area = 2.0000 in^2
Ixx = 0.1667 in^4
Iyy = 0.6667 in^4
Jz = 0.8333 in^4
Pxy = −0.0000 in^4
rxx = 0.2887 in
ryy = 0.5774 in
Sxx max = 0.3333 in^3
Sxx min = 0.3333 in^3
Syy max = 0.6667 in^3
Syy min = 0.6667 in^3



SECTION



FRONT VIEW

PALTE 3"x3"x ¼"
STEEL PLATE, FY=36KSI
(Properties of SECTION)

Area = 0.7500 in^2
Ixx = 0.5625 in^4
Iyy = 0.0039 in^4
Jz = 0.5664 in^4
Pxy = 0.0000 in^4
rxx = 0.8660 in
ryy = 0.0722 in
Sxx max = 0.3750 in^3
Sxx min = 0.3750 in^3
Syy max = 0.0313 in^3
Syy min = 0.0312 in^3

D. Mechanical Properties: Meet or exceed the following when tested in accordance with ASTM D143:

1. Bending Strength: 22,445 psi
2. Modulus of Elasticity: 3,145,000 psi/
3. Compression Parallel to Grain: 13,140 psi.
4. Compression Perpendicular to Grain: 3,595 psi.
5. Average Air-Dry Density: Ranges from 56.7 to 59.3pcf.
6. Basic Specific Gravity: Ranges from 0.85-0.97.

Fb= 5611.25 psi

Fb= 5611.25 psi

E. Coefficient of Friction: Meet or exceed the Static Coefficient of Friction for both Neolite and Leather in accordance with ASTM C1028 standard testing method:

1. Neolite: Dry .55 FP / Wet .79 FP.
2. Leather: Dry .73 FP / Wet .69 FP.

F. Decay Resistance: Meet or exceed International Building Code/International Residential Code requirements for "Naturally Durable Wood" being durability rated by the U.S Forest Products Laboratory or the Forest Research Laboratory Oregon State University as Class 2 or better. Ipe (Tabebuia spp, Lapacho Group) – Class 1.

G. Screw Pull Out: Meet or exceed a minimum average screw pull out of 1102 pounds at maximum load.

H. Fire Rating, Acute Inhalation, Combustion Toxicity Requirements: Meet or exceed the following.

1. Lumber supplied shall be naturally fire resistant without the use of any fire resistant treatments to meet NFPA Class A, 1989 edition, NFPA Class B, 2007 edition in accordance with ASTM E-84 standard test methods.
2. Lumber supplied shall be approved by CalFire Wildlife Urban Interface Areas – Materials and Construction Methods, Exterior Wildlife Exposure: Decking Code Section Chapter 7A(CSFM 12-7A-4).
3. Lumber supplied shall be approved as per San Francisco Building Code Section 1511.5 (rooftop decks).
4. Lumber supplied shall be approved as per City of NY Dept. of buildings, Fire Retardant Wood Code Sections 27-328 and carry a valid MEA#.
5. Lumber Supplied shall meet International Building Code / International Residential Code minimum requirements for "Fire Resistant Wood".
6. Lumber Supplied shall have an Acute Inhalation Combustion Toxicity testing result of LC 50 of 63.60g. (Pass (19.7g or greater) as per NYS Modified Pittsburg Protocol NYSUFPBC, Art. 15, Part 1120,9 NYCRR 1120.

I. Environmental Compliance:

1. Green by Nature Compliance: All lumber shall meet minimum environmental requirements as defined under Green By Nature…Build With Conscience™ Controlled Wood, Chain of Custody, Life Cycle Impact and Due Care – Environmental Compliance Standards, Policies and Procedures confirming that:

   a. All Iron Woods products have been verified of legal origin and compliance as being, legally harvested, transported, exported, imported and documented in compliance with all country of origin, international and domestic laws, rules, regulations and treaties pertaining to the fair and legal trade of forest products including but not limited to the U.S. Department of Agriculture Lacey Act, ITTA (International Tropical Timber Trade Agreement), CITES (Convention On The International Trade of Endangered Species), and U.S. Buy American Act.
   b. All Iron Woods products are derived from forests which are naturally occurring, renewable and sustainable and are not harvested from forests or forest plantations where traditional or civil rights have been violated, forests having high conservation values which are threatened, forests that have been genetically modified or forests which have been converted to non-forest use. All Iron Woods and their packaging materials have been kiln dried and or fumigated and are free from live and or invasive insect, plant or animal species.

Handrail & Circular Design Bracket Screws Located at the Border

Allowable Stresses for **Aluminum**				
	6063-T6 (psi)	6063-T5 (psi)	6061-T6 (psi)	6005-T5 (psi)
Fb (Tubes & Shapes)	15000	9500	19000	19000
	4800	4800	9000	8000
Fb (Round & Oval)	18000	11500	24000	24000
	5500	5500	10500	9000
Fv Tubes, Shapes & Round	8500	5500	12000	12000
	2800	2800	5000	4500
E	10100 ksi	10100 ksi	10100 ksi	10100 ksi

White bars apply to nonwelded members and to welded members at loc. fathers than 1.0 in from a weld
Shadded bars apply to within 1.0 in of a weld

Allowable Stress for Aluminum Welding:
Aluminum Filler Alloy 5356 Fv=7000 psi

Allowable Stress for Steel
Strong and Weak Axis Allowables Stress for Tube **Steel** Shape:
Fb=23760 psi, Fv=14400 psi for Fy=36 ksi E=29000 Ksi
Fb=33000 psi, Fv=20000 psi for Fy=50 ksi E=29000 Ks

Allowable Stress for Steel Welding:
Welding: E-70XX Fv=21000 psi

Allowable Stress for Stainless Steel
Strong and Weak Axis Allowables Stress for Tube Steel Shape:
Fb=16216 psi, Fv=9189 psi for (SS 304) Fy=30 ksi E=28000 Ksi
Fb=34730 psi, Fv=20838 psi for (S32304 Duplex SS) Fy=58 ksi E=28000 Ksi
Fb=62870 psi, Fv=37720 psi for (S17400-H1150) Fy=105 ksi E=28000 Ksi

Allowable Stress for Steel Welding:
Welding: E318 or E309 Fv=19200 psi

Allowable Stress for Wood
Strong and Weak Axis Allowables Stress for Lumber No.2 Non Dense (2"-4" thick):
Fb=1350 psi, Fv=175 psi for 2"-4" Wide E=1400 Ksi
Fb=1150 psi, Fv=175 psi for 5"-6" Wide E=1400 Ksi

DESIGN INFORMATION:

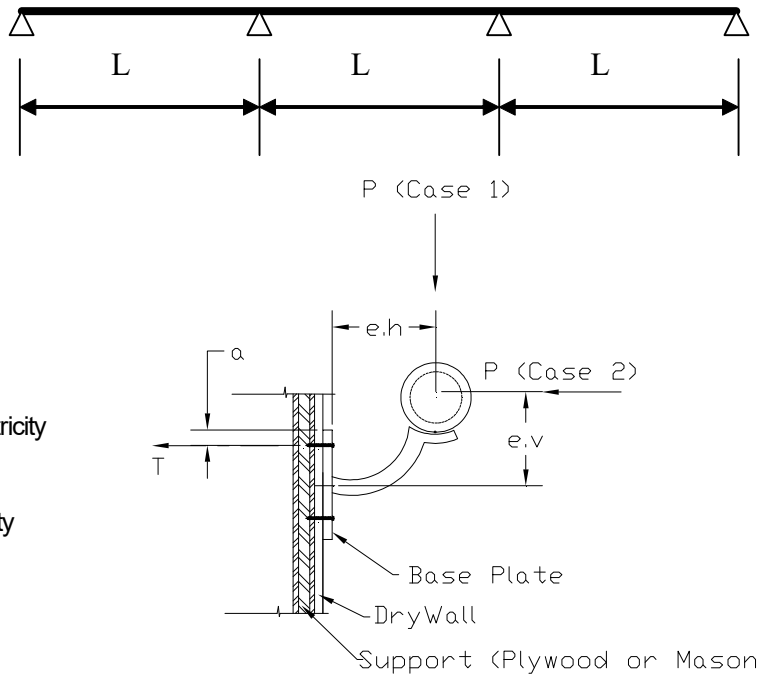
Florida Building Code 2023
AISC Manual of Steel Construction 16th Ed, NDS-ASD for Wood Construction 2015, ASCE 7-22
Structural Stainless Steel, Aluminum 2020

DESIGN LOADS:

Handrail Design Loads: Concentrated Load = 200 lbs. applied at any point in any direction.
Uniform Load = 50 plf applied in any direction (other than dwelling units).

Guardrail System Loads: Concentrated Load = 200 lbs. applied at any point in any direction at the top of the guardrail (other than dwelling units include a load of 100 lbs applied vertically downward at the top of guardrail).
Concentrated horizontal load of 200 lbs applied on a 1 sq ft. area at any point in the system.

Handrail & Bracket Design:



General Data:

$L := 36.00$ Railing Span (in)

$e_H := 3.25$ Horizontal Eccentricity of Loads (in)

$e_V := 3.00$ Vertical Eccentricity of Loads (in)

Handrail Data:

$Fb_{top} := 5600.00$ psi

$Fv_{top} := 175.00$ psi

$Sx_{top} := 0.333$ For Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.667$ For Horizontal Loads $\left(in^3\right)$

$A_{top} := 2.00$

Bracket Data:

$Fb_{bracket} := 21600.00$ psi

$Fv_{bracket} := 14400.00$ psi

$Sx_{bracket} := 0.187\ in^3$

$A_{bracket} := 1.50\ in^2$

Allowable Support Stress Data:

$Fp_{supp} := 1500.00$

Allowable compressive strength of support (psi)
0.35*f'c.....for Full Area of Concrete Support
0.35*f'c*SQR(A2/A1) < or = 0.7*f'c.....for less than Full Area of Concrete Support
0.7*f'm.....for Full Area Concrete Block Support
Fc.....for Wood Lumber or Plywood

Handrail Design:

Actual Moment:

Concentrated Load = 200 lbs.

$$Mpmax := \frac{200 \cdot L}{5}$$

Mpmax = 1440.00 in − lb

Uniform Load = 50 plf

$$M\omega max := 0.1012 \cdot \left(\frac{50}{12}\right) \cdot L^2$$

Mωmax = 546.48 in − lb

$$Mmax := \max(Mpmax, M\omega max)$$

Mmax = 1440.00 in − lb

Actual Shear:

Concentrated Load = 200 lbs.

$$Vpmax := 200\,lbs$$

Vpmax = 200.00 lbs

Uniform Load = 50 plf

$$V\omega max := 0.6 \cdot \left(\frac{50}{12}\right) \cdot L$$

Vωmax = 90 lbs

$$Vmax := \max(V\omega max, Vpmax)$$

Vmax = 200.00 lbs

Section Required:

Bending Design:

Section Modulus Required

$$Shr := \frac{Mmax}{Fb_{top}}$$

Shr = 0.26 in^3

Shear Design:

Area Required

$$Ahr := \frac{1.5 \cdot Vmax}{Fv_{top}}$$

Ahr = 1.71 in^2

Section Provided:

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

$BENDING_{top}$ = "OK"

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

$SHEAR_{top}$ = "OK"

Bracket Design:

Actual Reaction:

Concentrated Load = 200 lbs.

$Rpmax := 200.00$ lbs

Uniform Load = 50 plf

$R\omega max := 1.0 \cdot \left(\frac{50}{12}\right) \cdot L$

$R\omega max = 150.00$ lbs

$Rmax := \max(R\omega max, Rpmax)$

$Rmax = 200.00$ lbs

Actual Moment:

$Mbracket := Rmax \cdot \max(e_H, e_V)$

$Mbracket = 650.00$ in - lb

Actual Shear (For Vertical Load):

$Vbracket := Rmax$

$Vbracket = 200.00$ lbs.

Actual Axial (For Horizontal Load):

$Nbracket := Rmax$

$Nbracket = 200.00$ lbs.

Section Required:

Bending Design:

Section Modulus Required:

$$Sbracketreq := \frac{Mbracket}{Fb_{bracket}}$$

$Sbracketreq = 0.03$ in^3

Shear Design:

Area Required:

$$Abracketreq := \frac{1.5 \cdot Vbracket}{Fv_{bracket}}$$

$Abracketreq = 0.02$ in^2

Section Provided:

$BENDING_{bracket} := \text{if}(Sbracketreq \geq Sx_{bracket}, \text{"N.G"}, \text{"OK"})$

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

$SHEAR_{bracket} := \text{if}(Abracketreq \geq A_{bracket}, \text{"N.G"}, \text{"OK"})$

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

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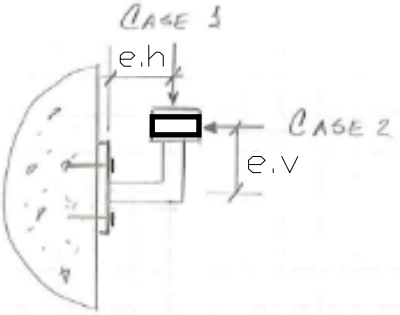
HANDRAIL CONNECTION TO CMU WALL



LOADS:

LL = 200 #

LOADS PROCESSING:

CASE 1

$M = 200\# * 3.25 = 650\#$

$T_{PER\ bolT} = \frac{650\#}{0.9 * 2.5"} = 289\#$

$T_{PER\ bolT} = 289\# + \frac{200\#}{2\ bolT} = 389\#$ WORST CASE

CONSERVATIVE ANALYSIS

$SHEAR_{PER\ bolT} = \frac{200\#}{2} = 100\#$

CASE 2

$M = 200\# * 3.00 = 600\#$

$T_{PER\ bolT} = \frac{600\#}{0.9 * 2.5"} = 267\#$ Worst case

Using: 1/4" GALVANIZED STEEL SCREW-BOLT + BY DEWALT W/2-1/2 EMBED, 2" SPACING AND 4" MIN. EDGE DISTANCE

See Spreadsheet printout for anchors capacity

T allow = 605# > 389# (OK)

S allow = 505# > 100# (OK)

COMBINED
T/T allow + S/S allow < 1
0.84 < 1 (OK)

Allowable Screw-Bolt+ Tension and Shear Load Capacities Installed into the face of Grout-Filled Concrete Masonry Units[1,2,3,4,5,6,7,8,9]

CODE LISTED
ICC-ES ESR-4042

Tension Load

Anchor Diameter, d in.	Minimum Embedment h_{nom} in. (mm)	Allowable Load at c_{cr} and s_{cr} lbs (kN)	Spacing Distance, s			Edge or End Distance, c_2 or c_1 (see Illustration of Screw-Bolt+ Installed into Grouted Concrete Masonry Wall detail)		
			Critical Distance, s_{cr} in. (mm)	Minimum Distance, s_{min} in. (mm)	Allowable Load Factor at s_{min}	Critical Distance, c_{cr} in. (mm)	Minimum Distance, c_{min} in. (mm)	Allowable Load Factor at c_{min}
1/4	1-5/8 (41.3)	315 (1.4)	4 (101.6)	2 (50.8)	1.00 (no reduction)	3-3/4 (95.3)	1-1/4 (31.8)	0.60
	2-1/2 (63.5)	605 (2.7)						
3/8	2 (50.8)	450 (2.0)	6 (152.4)	3 (76.2)	1.00 (no reduction)	6 (152.4)	1-1/2 (38.1)	0.70
	3-1/4 (82.6)	1,085 (4.8)						
1/2	2-1/2 (63.5)	610 (2.7)	8 (203.2)	4 (101.6)	1.00 (no reduction)	8 (203.2)	2-5/8 (66.7)	0.75
	4-1/4 (108.0)	1,190 (5.3)						
5/8	3-1/4 (82.6)	880 (3.9)	10 (254.0)	4 (101.6)	1.00 (no reduction)	10 (254.0)	3-3/8 (85.7)	0.90
	5 (127.0)	1,270 (5.6)						
3/4	4 (101.6)	1,150 (5.1)	12 (304.8)	4 (101.6)	1.00 (no reduction)	12 (304.8)	4 (101.6)	1.00 (no reduction)
	6-1/4 (158.8)	1,355 (6.0)						

Shear Load

Anchor Diameter, d in.	Minimum Embedment h_{nom} in. (mm)	Allowable Load at c_{cr} and s_{cr} Direction 1 & 2 lbs[9] (kN)	Allowable Load at c_{cr} and s_{cr} Direction 3 & 4 lbs[9] (kN)	Spacing Distance, s			Edge or End Distance, c_2 or c_1 (see Illustration of Screw-Bolt+ Installed into Grouted Concrete Masonry Wall detail)			
				Critical Distance, s_{cr} in. (mm)	Minimum Distance, s_{min} in. (mm)	Allowable Load Factor at s_{min}	Critical Distance, c_{cr} in. (mm)	Minimum Distance, c_{min} in. (mm)	Allowable Load Factor at c_{min}	
									Load Perpendicular to Edge or End (Direction 1 & 2)[9]	Load Perpendicular to Edge or End (Direction 3 & 4)[9]
1/4	1-5/8 (41.3)	400 (1.8)	400 (1.8)	4 (101.6)	2 (50.8)	1.00 (no reduction)	3-3/4 (95.3)	1-1/4 (31.8)	0.35	1.00 (no reduction)
	2-1/2 (63.5)	505 (2.2)	505 (2.2)							
3/8	2 (50.8)	815 (3.6)	815 (3.6)	6 (152.4)	3 (76.2)	1.00 (no reduction)	6 (152.4)	1-1/2 (38.1)	0.27	1.00 (no reduction)
	3-1/4 (82.6)	935 (4.2)	935 (4.2)							
1/2	2-1/2 (63.5)	1,380 (6.1)	1,380 (6.1)	8 (203.2)	4 (101.6)	1.00 (no reduction)	8 (203.2)	2-5/8 (66.7)	0.20	1.00 (no reduction)
	4-1/4 (108.0)	2,180 (9.7)	2,180 (9.7)							
5/8	3-1/4 (82.6)	2,090 (9.3)	2,225 (9.9)	10 (254.0)	4 (101.6)	1.00 (no reduction)	10 (254.0)	3-3/8 (85.7)	0.23	1.00 (no reduction)
	5 (127.0)	2,640 (11.7)	2,640 (11.7)							
3/4	4 (101.6)	2,800 (12.5)	3,330 (14.8)	12 (304.8)	4 (101.6)	1.00 (no reduction)	12 (304.8)	4 (101.6)	0.25	1.00 (no reduction)
	6-1/4 (158.8)	3,100 (13.8)	3,685 (16.4)							

For SI: 1 inch = 25.4 mm; 1 lbs = 0.0044 kN, 1 psi = 0.006894 MPa.

1. All values are for anchors installed in fully grouted concrete masonry wall construction with materials meeting minimum compressive strength, f'm, of 1,500 psi (10.3 MPa). Concrete masonry units must be light-, medium, or normal-weight conforming to ASTM C90. Allowable loads are based on a safety factor of 5.0.
2. Anchors may be installed in any location in the face of the masonry wall (cell, web, bed joint) except within 1-1/4-inch from the of the vertical mortar joint (head joint), center-to-center, provided the minimum edge and end distances are maintained. Anchors may not be placed in the head joint unless the vertical joint is mortared full-depth.
3. A maximum of two anchors may be installed in a single masonry cell in accordance with the spacing and edge or end distance requirements. Embedment is measured from the outside surface of the concrete masonry unit to the embedded end of the anchor. See the Illustration of Screw-Bolt+ Anchors Installed into Grouted Concrete Masonry Wall figure.
4. The critical spacing distance, s_{cr}, is the anchor spacing where full load values in the table may be used. The minimum spacing distance, s_{min}, is the minimum anchor spacing for which values are available and installation is permitted. Spacing distance is measured from the centerline to centerline between two anchors.
5. The critical edge or end distance, c_{cr}, is the distance where full load values in the table may be used. The minimum edge or end distance, c_{min}, is the minimum distance for which values are available and installation is permitted. Edge or end distance is measured from anchor centerline to the closest unrestrained edge.
6. The tabulated values are applicable for anchors installed into the ends of grout-filled concrete masonry units (e.g. wall opening) where minimum edge distances are maintained.
7. Load values for anchors installed less than s_{cr} and c_{cr} must be multiplied by the appropriate load reduction factor based on actual spacing (s) or edge distance (c). Load factors are multiplicative; both spacing and edge reduction factors must be considered.
8. Linear interpolation of load values between minimum spacing (s_{min}) and critical spacing (s_{cr}) and between minimum edge or end distance (c_{min}) and critical edge or end distance (c_{cr}) is permitted.
9. See Illustration of Shear Loading Relative to Edge and End of Masonry Wall figure for illustration of shear load directions.

TECHNICAL GUIDE – MECHANICAL ANCHORS ©2020 DEWALT – REV. E

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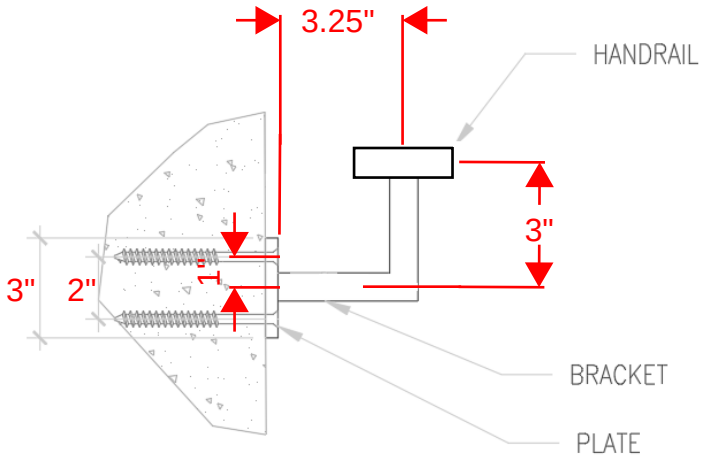
PLATE DESIGN



LOADS:

T= 389# (WORST CASE)

LOADS PROCESSING

CASE NO.2

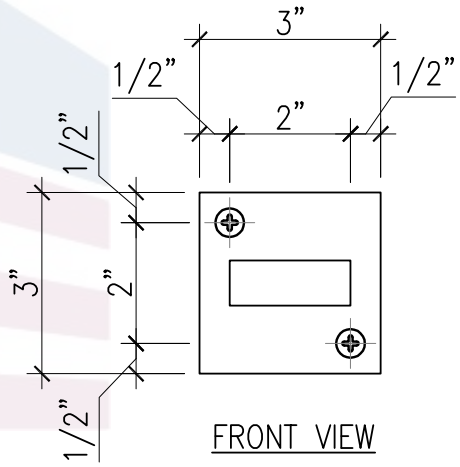
M= T * ec

M= 389# * 1.0" = 389#-in (conservative analysis)

$fb = \frac{M}{Sx} < Fb = 21600 \text{ psi}$

$Sx = \frac{b * t^2}{6} = \frac{3" * (1/4)^2}{6} = 0.03125 \text{ in}^3$

$fb = \frac{389\#\text{-in}}{0.03125 \text{ in}^3} = 12448 < Fb = 21600 \text{ psi}$



Using: 3"X3"X1/4" Steel Plate. W/ 2 BOLTS

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CABLE RAILINGS AT BALCONIES DESIGN

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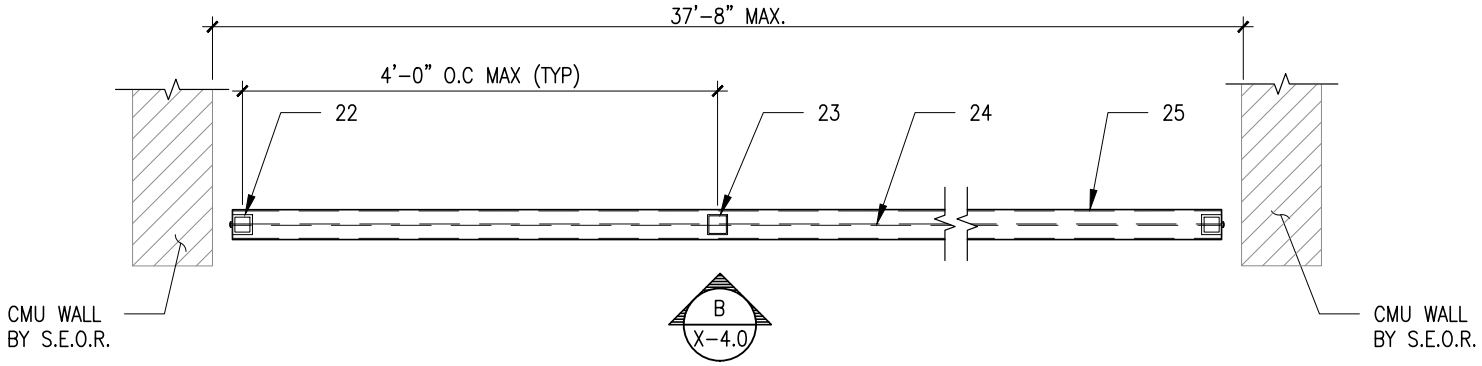
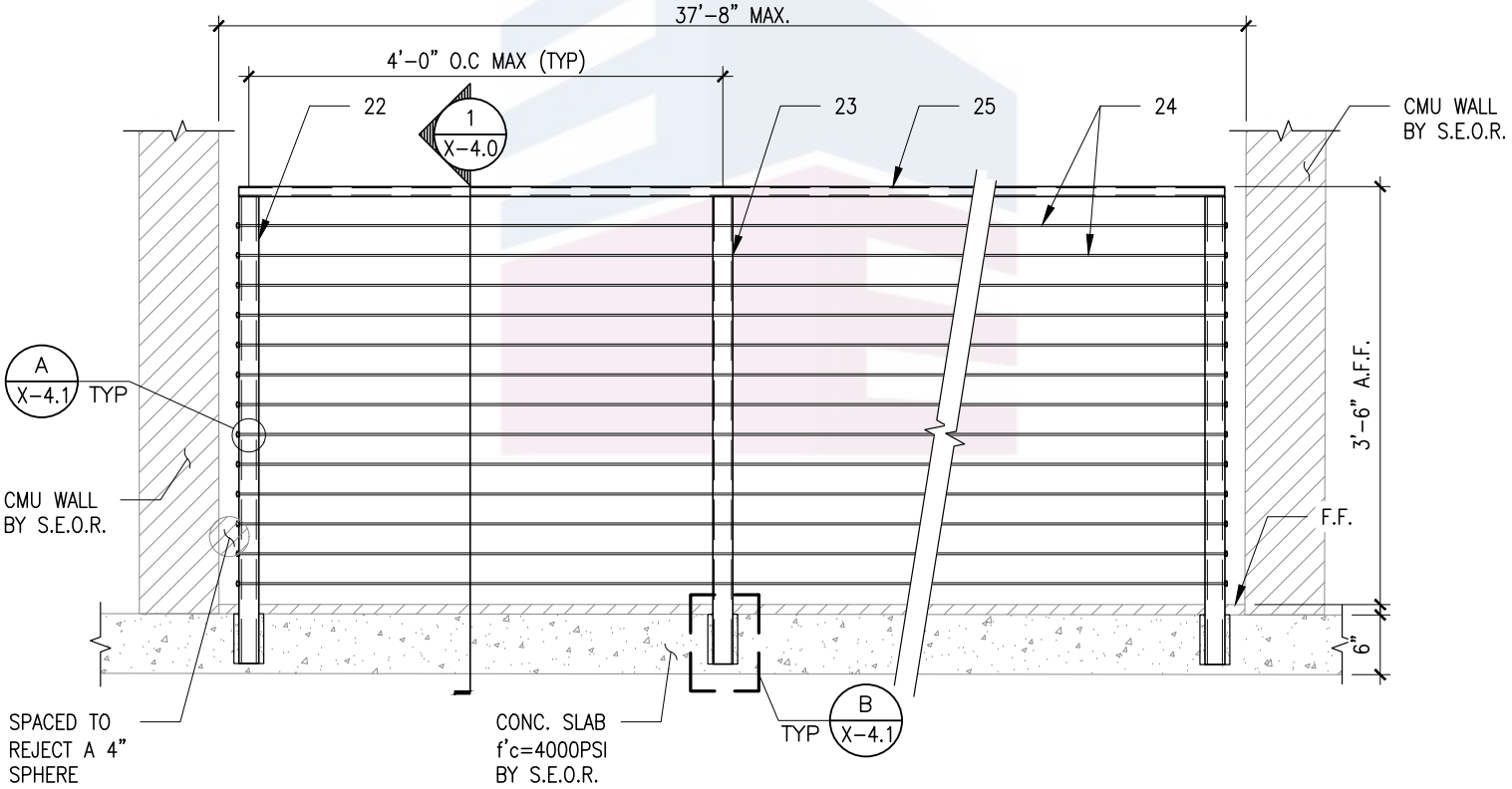


A X-4.0 CABLE RAILING PLAN VIEW

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



B X-4.0 CABLE RAILING ELEVATION

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

SKETCH

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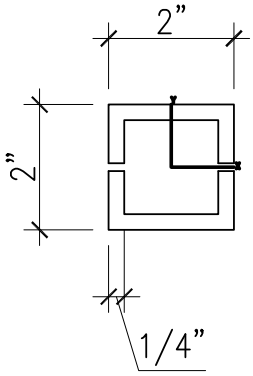
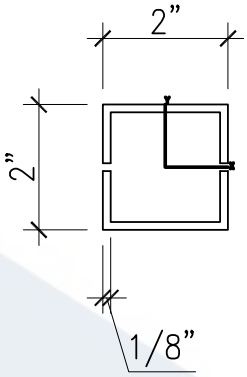
SECTION PROPERTIES



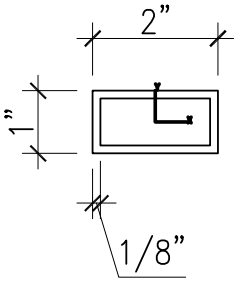
END POST RT2X2X¼"
ALLOY 6061-T6

Area = 1.6876 in^2
Ixx = 0.9114 in^4
Iyy = 0.8634 in^4
Jz = 1.7747 in^4
Pxy = -0.0000 in^4
rxx = 0.7349 in
ryy = 0.7153 in
Sxx max = 0.9114 in^3
Sxx min = 0.9114 in^3
Syy max = 0.8634 in^3
Syy min = 0.8634 in^3



INTERMEDIATE POST RT2X2X⅛"
ALLOY 6061-T6

Area = 0.9063 in^2
Ixx = 0.5517 in^4
Iyy = 0.5243 in^4
Jz = 1.0760 in^4
Pxy = 0.0000 in^4
rxx = 0.7802 in
ryy = 0.7606 in
Sxx max = 0.5517 in^3
Sxx min = 0.5517 in^3
Syy max = 0.5243 in^3
Syy min = 0.5243 in^3



TOP RAIL RT2X1X⅛"
ALLOY 6061-T6

Area = 0.6875 in^2
Ixx = 0.1051 in^4
Iyy = 0.3317 in^4
Jz = 0.4368 in^4
Pxy = -0.0000 in^4
rxx = 0.3911 in
ryy = 0.6946 in
Sxx max = 0.2103 in^3
Sxx min = 0.2103 in^3
Syy max = 0.3317 in^3
Syy min = 0.3317 in^3

Horizontal Cable Railing Design (Top, Post) (No Bottom)

Allowable Stresses for **Aluminum**				
	6063-T6 (psi)	6063-T5 (psi)	6061-T6 (psi)	6005-T5 (psi)
Fb	15000	9500	19000	19000
(Tubes & Shapes)	4800	4800	9000	8000
Fb	18000	11500	24000	24000
(Round & Oval)	5500	5500	10500	9000
Fv	8500	5500	12000	12000
Tubes, Shapes & Round	2800	2800	5000	4500
E	10100 ksi	10100 ksi	10100 ksi	10100 ksi

White bars apply to nonwelded members and to welded members at loc. fathers than 1.0 in from a weld
Shadded bars apply to within 1.0 in of a weld

Allowable Stress for Aluminum Welding:
Aluminum Filler Alloy 5356 Fv=7000 psi

Allowable Stress for Steel
Strong and Weak Axis Allowables Stress for Tube **Steel** Shape:
Fb=23760 psi, Fv=14400 psi for Fy=36 ksi E=29000 Ksi
Fb=33000 psi, Fv=20000 psi for Fy=50 ksi E=29000 Ks

Allowable Stress for Steel Welding:
Welding: E-70XX Fv=21000 psi

Allowable Stress for Stainless Steel
Strong and Weak Axis Allowables Stress for Tube Steel Shape:
Fb=16216 psi, Fv=9189 psi for (SS 304) Fy=30 ksi E=28000 Ksi
Fb=34730 psi, Fv=20838 psi for (S32304 Duplex SS) Fy=58 ksi E=28000 Ksi
Fb=62870 psi, Fv=37720 psi for (S17400-H1150) Fy=105 ksi E=28000 Ksi

Allowable Stress for Steel Welding:
Welding: E318 or E309 Fv=19200 psi

Allowable Stress for Wood
Strong and Weak Axis Allowables Stress for Lumber No.2 Non Dense (2"-4" thick):
Fb=1350 psi, Fv=175 psi for 2"-4" Wide E=1400 Ksi
Fb=1150 psi, Fv=175 psi for 5"-6" Wide E=1400 Ksi

DESIGN INFORMATION:

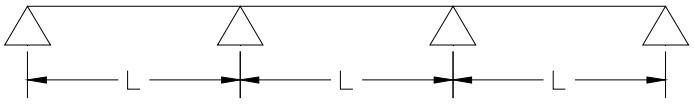
Florida Building Code 2023
AISC Steel Construction Manual 2017, NDS-2018 National Design Specifications for Wood Construction, ASCE/SEI 7-22, ASCE/SEI 8-02 Stainless Steel, Aluminum Design Manual 2020

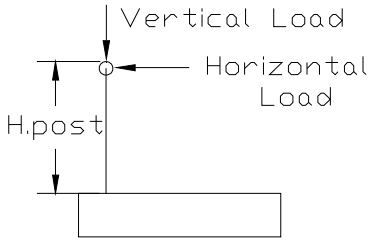
DESIGN LOADS:

Handrail Design Loads: Concentrated Load = 200 lbs. applied at any point in any direction.
Uniform Load = 50 plf applied in any direction (other than dwelling units).

Guardrail System Loads: Concentrated Load = 200 lbs. applied at any point in any direction at the top of the guardrail (other than dwelling units include a load of 100 lbs applied vertically downward at the top of guardrail).
Concentrated horizontal load of 50 lbs applied on a 1 sq ft. area at any point in the system.

Handrail Design:





General Data:

$L_{span} := 48.00$ Railing Span (in)

$H_{post} := 42.00$ Post Heigh (in)

General Loads:

$q_{wind} := 10.00$ Wind Pressure (psf)

$P_{200} := 200.00$ Single Concentrated Load (lbs) .....Railing P=200.00, Fence P=0.00

$q_{50} := 50.00$ Uniform Distributed Load (plf).....Railing q=50.00, Fence q=0.00

Top Railing Data:

$Fb_{TR} := 19000.00$ psi

$Fv_{TR} := 5000.00$ psi

$Sx_{TR} := 0.3067$ For Vertical Loads $\left(in^3\right)$

$Sy_{TR} := 0.6335$ For Horizontal Loads $\left(in^3\right)$

$A_{TR} := 0.9376$ in^2

Cable Data:

$Fy_{cable} := 50000.00$ psi

$\phi_{cable} := 0.125$ in

$E_{cable} := 16345000.00$ psi

Post Anchor Data:

$Fb_{PT} := 19000.00$ psi

$Fv_{PT} := 5000.00$ psi

$Sx_{PT} := 0.9114$ For Railings Loads $\left(in^3\right)$

$Sy_{PT} := 0.8633$ For Cable Loads $\left(in^3\right)$

$A_{PT} := 1.6875$ in^2

Post Intermediate Data:

$Fb_{PT.I} := 19000.00$ psi

$Fv_{PT.I} := 12000.00$ psi

$Sx_{PT.I} := 0.5517$ For Railings Loads $\left(in^3\right)$

$A_{PT.I} := 0.9062$ in^2

Cable Design:

$L_{cable} := 48.00$ Actual Cable Intermediate Post Span (in)

$s_{cable} := 3.00$ Cable spacing O.C.(in)

$L_T := 460.00$ Length of cable between anchor points (in)

$F_{ps} := 165.00$ Applied prestressing Force (lbs)

For Sphere Pass-Through Resistance with
w=50 psf through 2 cables & Vertical Actual Deflection Plus Impact Factor=1

$$A_{cable} := \pi \cdot \frac{\phi_{cable}^{2}}{4}$$

$A_{cable} = 0.01$ in^2

$$I_{cable} := 0.70\left(\pi \cdot \frac{\phi_{cable}^{4}}{64}\right)$$

$I_{cable} = 0.00$ in^4

$$\Delta_V := \left| \begin{array}{l} x \leftarrow 0.01 \\ \text{while } \frac{4 \cdot x \cdot E_{cable} \cdot A_{cable}}{L_{cable}} \cdot \frac{\sqrt{4 \cdot x^2 + L_{cable}^{2}} - L_{cable}}{\sqrt{4 \cdot x^2 + L_{cable}^{2}} + L_T - L_{cable}} + \frac{48 \cdot E_{cable} \cdot I_{cable} \cdot x}{L_{cable}^{3}} + \frac{4 \cdot F_{ps} \cdot x}{L_{cable}} \leq \frac{\left(50 \cdot \frac{\pi \cdot 4^2}{4 \cdot 144}\right)}{2 \cdot \cos\left(\operatorname{asin}\left(\frac{s_{cable} + 2 \cdot x}{4 + \phi_{cable}}\right)\right)} \\ x \leftarrow x + 0.001 \end{array} \right.$$

$\Delta_V = 0.36$ in

$$P_{ef} := \frac{\left(50 \cdot \frac{\pi \cdot 4^2}{4 \cdot 144}\right)}{2 \cos\left(\operatorname{asin}\left(\frac{s_{cable} + 2 \cdot \Delta_V}{4 + \phi_{cable}}\right)\right)}$$

$P_{ef} = 5.08$ lbs

$$T_{cable} := \frac{P_{ef}}{2 \cdot \Delta_V} \cdot \sqrt{\Delta_V^{2} + \frac{L_{cable}^{2}}{4}}$$

$T_{cable} = 168.11$ lbs

Cable Section Required:

$$\phi_{cable.req} := \sqrt{4 \cdot \frac{\frac{T_{cable}}{0.6 \cdot Fy_{cable}}}{\pi}}$$

$\phi_{cable.req} = 0.08$ in

Section Provided:

$TENSION_{cable} := \text{if}\left(\phi_{cable} \geq \phi_{cable.req}, \text{"O.K."}, \text{"N.G."}\right)$

$TENSION_{cable} = \text{"O.K."}$

Top Railing Vertical Design:

Actual Moment for Vertical Loads:

Concentrated Load = 200 lbs

$$M_{TR.200.V} := \frac{P_{200} \cdot L}{5}$$

$M_{TR.200.V} = 1920.00$ in − lb

Uniform Load = 50 plf

$$M_{TR.50.V} := 0.1012 \cdot \left(\frac{q_{50}}{12}\right) \cdot L^2$$

$M_{TR.50.V} = 971.52$ in − lb

$$M_{TR.V} := \max\left(M_{TR.50.V}, M_{TR.200.V}\right)$$

$M_{TR.V} = 1920.00$ in − lb

Actual Shear for Vertical Loads:

Concentrated Load = 200 lbs.

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

$V_{TR.200} = 200.00$ lbs

Uniform Load = 50 plf.

$$V_{TR.50} := 0.6 \cdot \left(\frac{q_{50}}{12} \cdot L\right)$$

$V_{TR.50} = 120.00$ lbs

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

$V_{TR.V} = 200.00$ lbs

Actual Axial for Cable Loads:

$$P_{TR.V} := \frac{T_{cable} \cdot \text{floor}\left(\frac{H_{post}}{s_{cable}}\right)}{2}$$

$P_{TR.V} = 1176.75$ lbs.

***See Attached Axial and Bending Combined Calculations for Top Railing Vertical Loads

Top Railing Horizontal Design:

Actual Moment for Horizontal Loads:

Concentrated Load = 200 lbs

$$M_{TR.200.H} := \frac{P_{200} \cdot L}{5}$$

$M_{TR.200.H} = 1920.00$ in – lb

Uniform Load = 50 plf

$$M_{TR.50.H} := 0.1012 \cdot \left(\frac{q_{50}}{12}\right) \cdot L^2$$

$M_{TR.50.H} = 971.52$ in – lb

For Wind Pressure

$$M_{TR.wind} := 0.1012 \cdot \left(\frac{q_{wind}}{144}\right) \cdot \left(\frac{H_{post}}{2} \cdot L^2\right)$$

$M_{TR.wind} = 340.03$ in – lb

$$M_{TR.H} := \max\left(M_{TR.200.H}, M_{TR.50.H}, M_{TR.wind}\right)$$

$M_{TR.H} = 1920.00$ in – lb

Actual Shear for Horizontal Loads:

Concentrated Load = 200 lbs.

$$V_{TR.200.H} := P_{200}$$

$V_{TR.200.H} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{TR.50.H} := 0.6 \cdot \left(\frac{q_{50}}{12}\right) \cdot L$$

$V_{TR.50.H} = 120$ lbs

For Wind Pressure

$$V_{TR.wind} := 0.6 \cdot \left(\frac{q_{wind}}{144}\right) \cdot \left(\frac{H_{post}}{2}\right) \cdot L$$

$V_{TR.wind} = 42.00$ lbs

$$V_{TR.H} := \max\left(V_{TR.200.H}, V_{TR.50.H}, V_{TR.wind}\right)$$

$V_{TR.H} = 200.00$ lbs

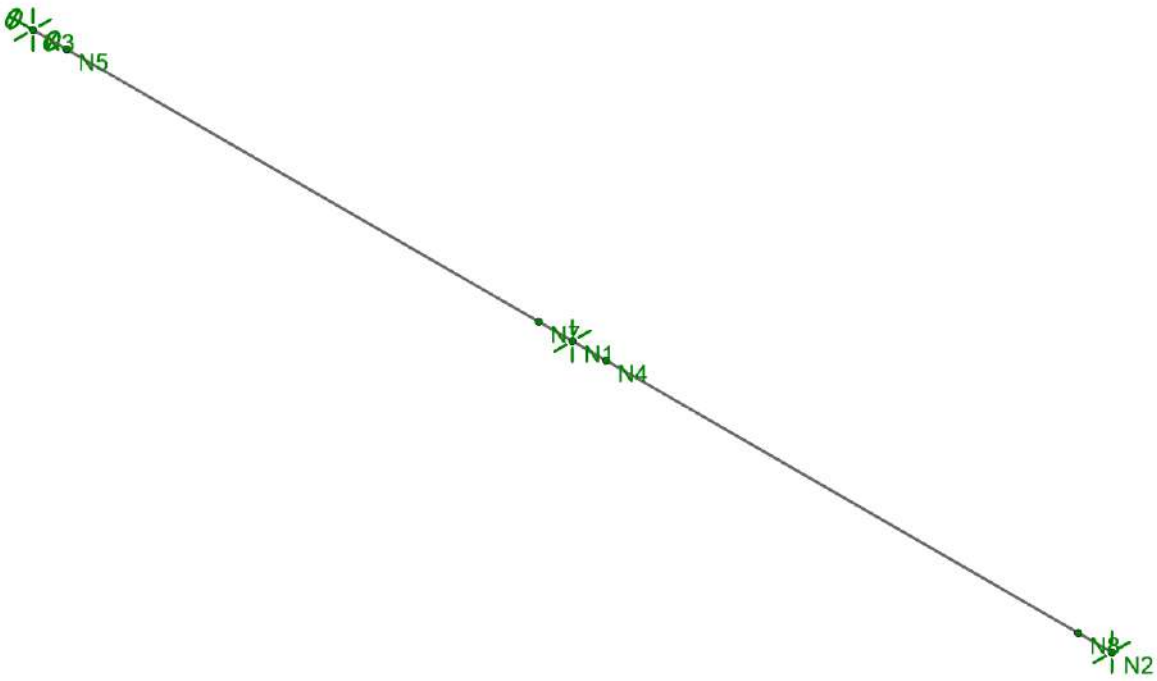
Actual Axial for Cable Loads:

$$P_{TR.H} := \frac{T_{cable} \cdot \text{floor}\left(\frac{H_{post}}{s_{cable}}\right)}{2}$$

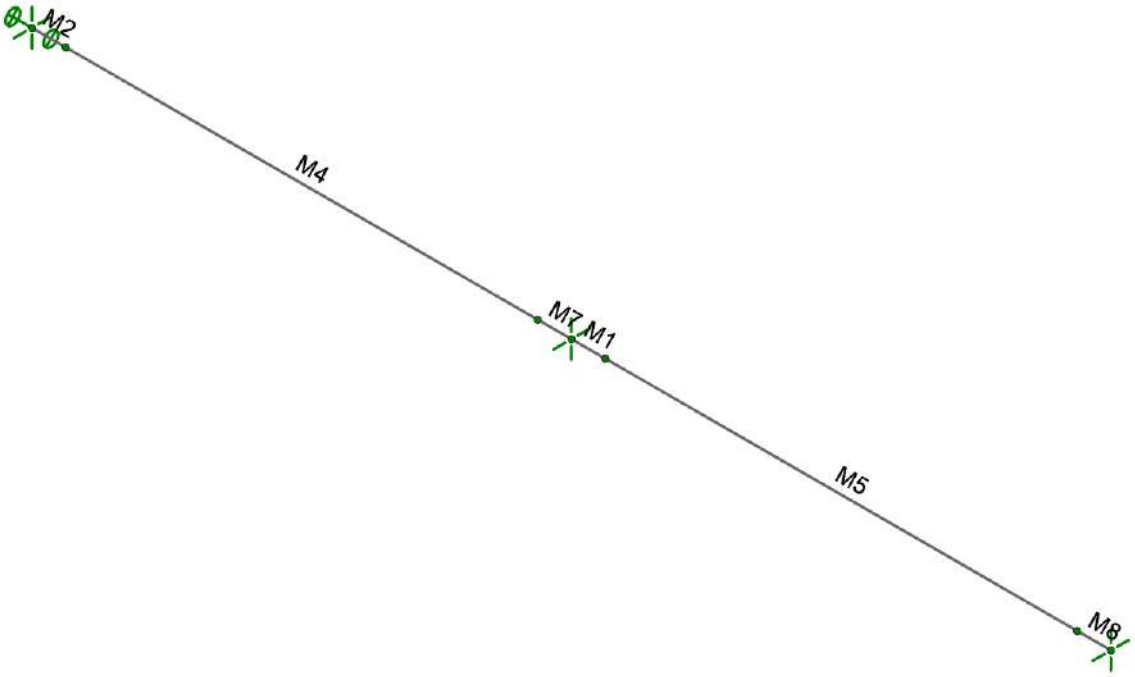
$P_{TR.H} = 1176.75$ lbs.

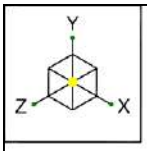
***See Attached Axial and Bending Combined Calculations for Top Railing Horizontal Loads

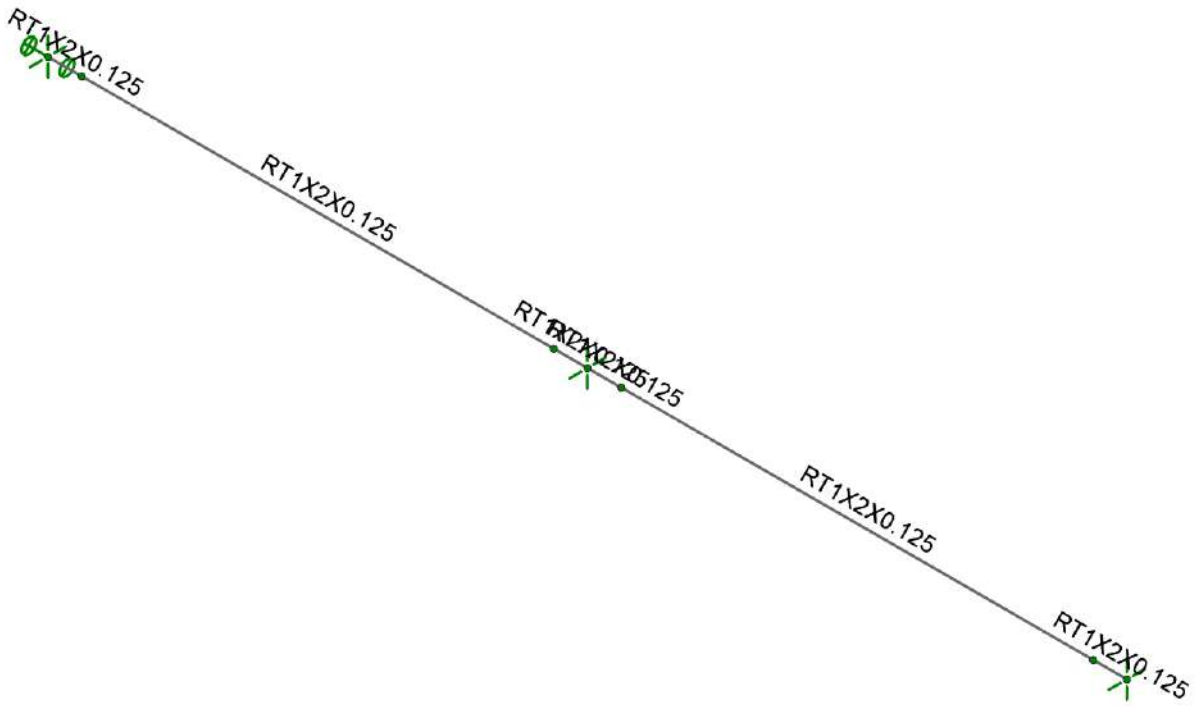
TOP RAIL
COMBINED AXIAL & BENDING DESIGN

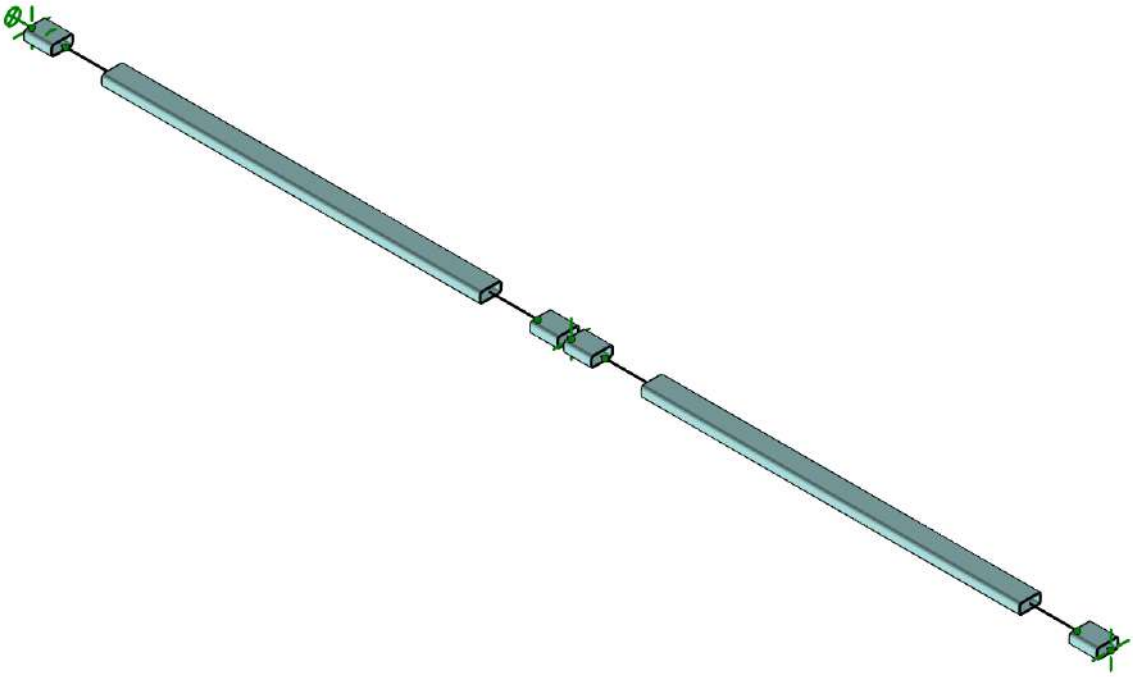


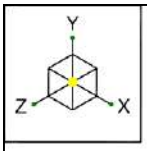
IRISA	EasternEngineering		SK-1
	M.B		Jul 17, 2025 at 05:24 PM
			top rail v00.r3d

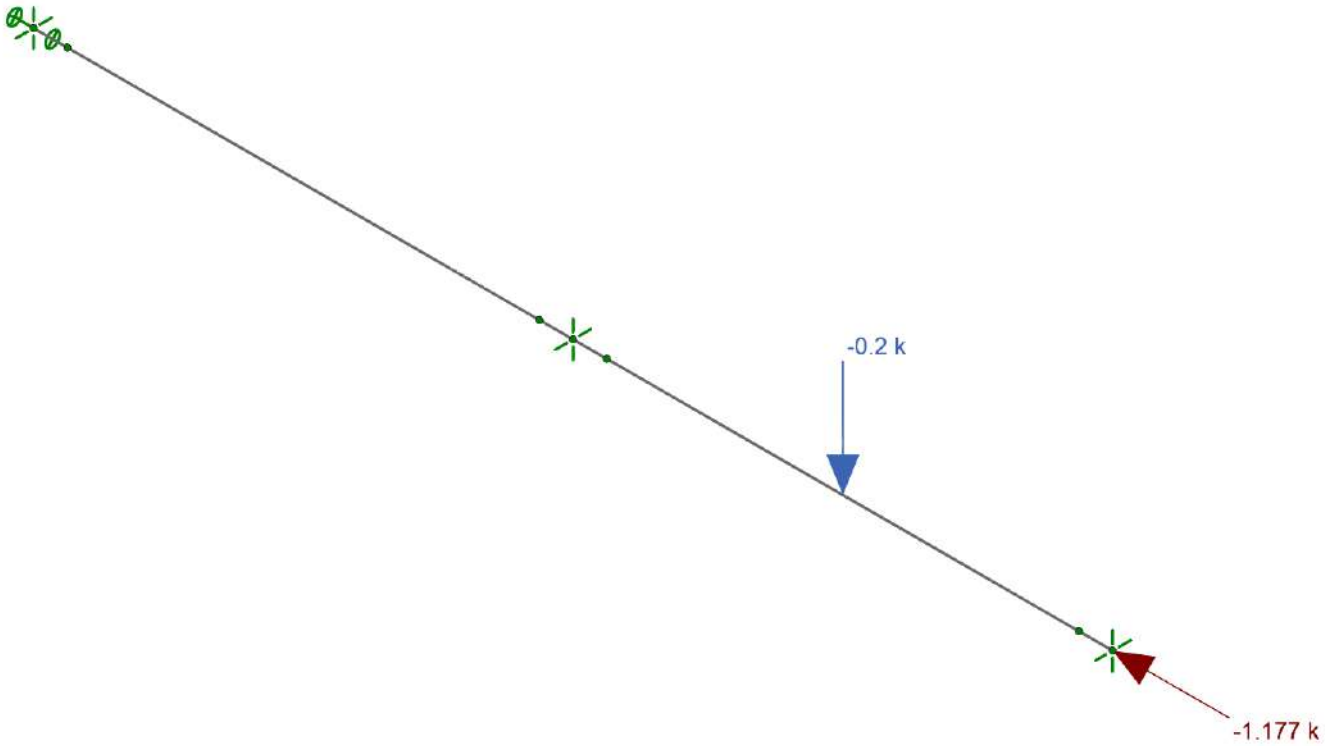

M2
M4
M7
M1
M5
M8


Y
Z
X


RT1X2X0.125
RT1X2X0.125
RT1X2X0.125
RT1X2X0.125

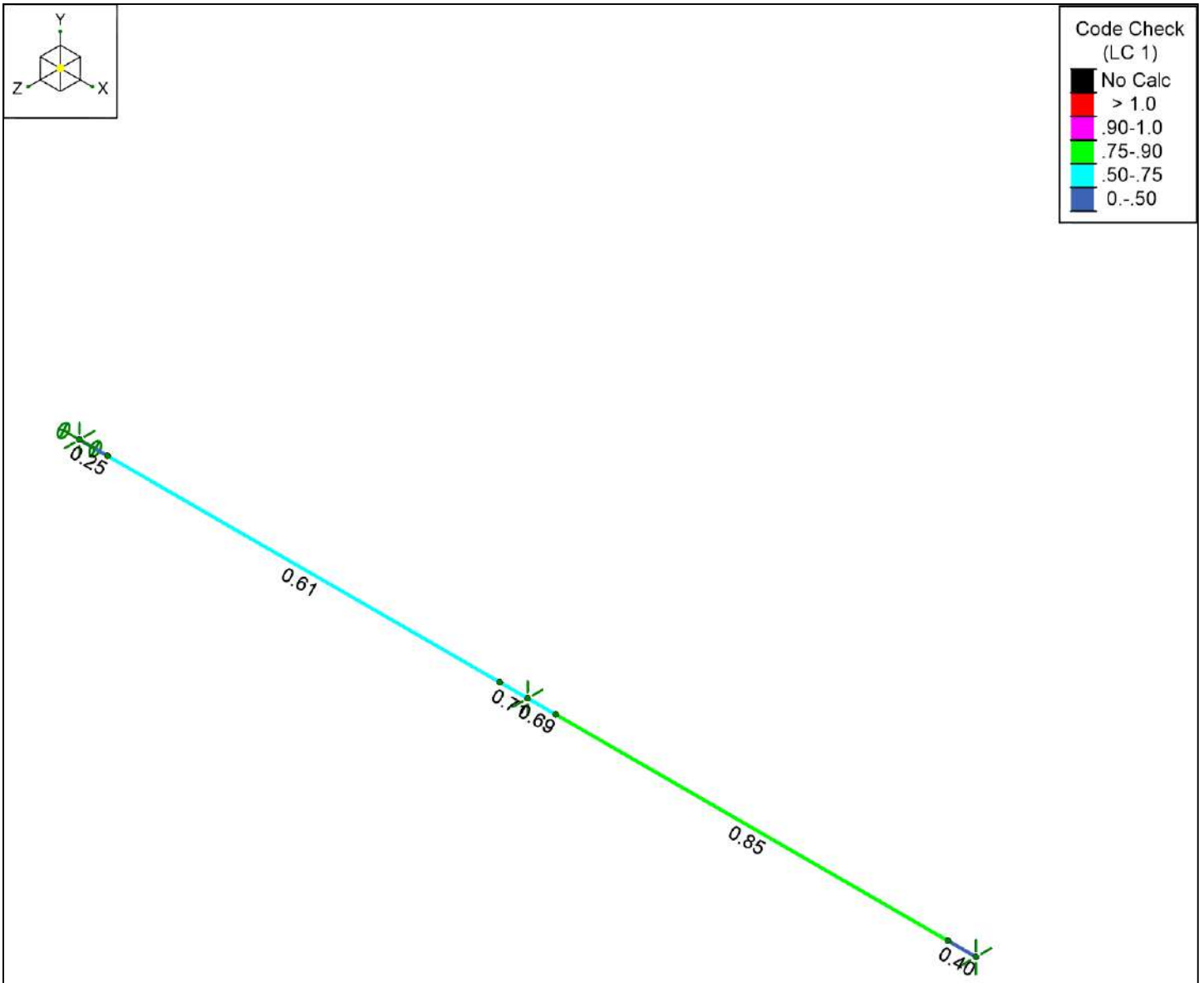






Loads: LC 1,

IRISA A NEMETSCHEK COMPANY	EasternEngineering		SK-5
	M.B		Jul 17, 2025 at 05:31 PM
			top rail v00.r3d



Member Code Checks Displayed
Results for LC 1,

RISA A NEMETSCHEK COMPANY	EasternEngineering		SK-6
	M.B		Jul 17, 2025 at 05:32 PM
			top rail v00.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	4	0	0	
3	N3	-4	0	0	
4	N4	0.25	0	0	
5	N5	-3.75	0	0	
6	N7	-0.25	0	0	
7	N8	3.75	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N1		Reaction	Reaction	
2	N2		Reaction	Reaction	
3	N3	Reaction	Reaction	Reaction	Reaction

Aluminum Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Top rail weld affected	RT1X2X0.125	Beam	Rectangular Tubes	6061-T6 W	Typical	0.688	0.105	0.332	0.245
2	Top rail not weld affected	RT1X2X0.125	Beam	Rectangular Tubes	6061-T6	Typical	0.688	0.105	0.332	0.245

Aluminum Design Parameters

	Label	Shape	Length [ft]	Lcomp top [ft]	Function
1	M1	Top rail weld affected	0.25	Lbyy	Lateral
2	M2	Top rail weld affected	0.25	Lbyy	Lateral
3	M4	Top rail not weld affected	3.5	Lbyy	Lateral
4	M5	Top rail not weld affected	3.5	Lbyy	Lateral
5	M7	Top rail weld affected	0.25	Lbyy	Lateral
6	M8	Top rail weld affected	0.25	Lbyy	Lateral

Aluminum Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Table B.4	kt	Ftu [ksi]	Fty [ksi]	Fcy [ksi]	Fsu [ksi]	Ct
1	3003-H14	10100	3787.5	0.33	1.3	0.173	Table B.4-1	1	19	16	13	12	141
2	6061-T6	10100	3787.5	0.33	1.3	0.173	Table B.4-2	1	38	35	35	24	141
3	6063-T5	10100	3787.5	0.33	1.3	0.173	Table B.4-2	1	22	16	16	13	141
4	6063-T6	10100	3787.5	0.33	1.3	0.173	Table B.4-2	1	30	25	25	19	141
5	5052-H34	10200	3787.5	0.33	1.3	0.173	Table B.4-1	1	34	26	24	20	141
6	6061-T6 W	10100	3787.5	0.33	1.3	0.173	Table B.4-1	1	24	15	15	15	389

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	0	0.143	0	0	0	0
2	1	N2	0	0.082	0	0	0	0
3	1	N3	1.177	-0.018	0	0	0	0
4	1	Totals:	1.177	0.207	0			
5	1	COG (ft):	X: 1.936	Y: 0	Z: 0			

Node Displacements

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	-0.01	0	0	0	0	-1.996e-2
2	1	N2	-0.02	0	0	0	0	2.938e-2
3	1	N3	0	0	0	0	0	1.054e-2
4	1	N4	-0.011	-0.065	0	0	0	-2.246e-2
5	1	N5	-0.001	0.032	0	0	0	1.034e-2
6	1	N7	-0.01	0.055	0	0	0	-1.652e-2
7	1	N8	-0.02	-0.088	0	0	0	2.865e-2

AA ADM1-20: ASD - BUILDING Member Aluminum Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	Pnc/Om[k]	Pnt/Om[k]	Mny/Om[k-ft]	Mnz/Om[k-ft]	Vny/Om[k]	Vnz/Om[k]	Cb	Eqn
1	1	M1	RT1X2X0.125	0.687	0	0.19	0	z	6.255	6.255	0.159	0.252	2.216	0.852	1	H.1-1
2	1	M2	RT1X2X0.125	0.248	0.25	0.045	0.25	z	6.255	6.255	0.159	0.252	2.216	0.852	1	H.1-1
3	1	M4	RT1X2X0.125	0.608	3.5	0.012	3.5	z	3.057	13.407	0.371	0.587	5	1.923	1	H.1-1
4	1	M5	RT1X2X0.125	0.854	1.75	0.064	0	z	3.057	13.407	0.371	0.587	5	1.923	1	H.1-1
5	1	M7	RT1X2X0.125	0.707	0	0.015	0	z	6.255	6.255	0.159	0.252	2.216	0.852	1	H.1-1
6	1	M8	RT1X2X0.125	0.403	0	0.161	0.25	z	6.255	6.255	0.159	0.252	2.216	0.852	1	H.1-1

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	1.177	0	-0.162	0	0.079	0
2			2	1.177	0	-0.162	0	0.069	0
3			3	1.177	0	-0.162	0	0.059	0
4			4	1.177	0	-0.162	0	0.049	0
5			5	1.177	0	-0.162	0	0.039	0
6	1	M2	1	1.177	0	0.038	0	0	0
7			2	1.177	0	0.038	0	0.002	0
8			3	1.177	0	0.038	0	0.005	0
9			4	1.177	0	0.038	0	0.007	0
10			5	1.177	0	0.038	0	0.01	0
11	1	M4	1	1.177	0	0.019	0	0.01	0
12			2	1.177	0	0.02	0	0.027	0
13			3	1.177	0	0.021	0	0.045	0
14			4	1.177	0	0.022	0	0.063	0
15			5	1.177	0	0.022	0	0.083	0
16	1	M5	1	1.177	0	-0.122	0	0.039	0
17			2	1.177	0	-0.122	0	-0.068	0
18			3	1.177	0	0.079	0	-0.174	0
19			4	1.177	0	0.08	0	-0.104	0
20			5	1.177	0	0.081	0	-0.034	0
21	1	M7	1	1.177	0	-0.013	0	0.083	0
22			2	1.177	0	-0.013	0	0.082	0
23			3	1.177	0	-0.013	0	0.081	0
24			4	1.177	0	-0.013	0	0.08	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
25			5	1.177	0	-0.013	0	0.079	0
26	1	M8	1	1.177	0	0.137	0	-0.034	0
27			2	1.177	0	0.137	0	-0.026	0
28			3	1.177	0	0.137	0	-0.017	0
29			4	1.177	0	0.137	0	-0.009	0
30			5	1.177	0	0.137	0	0	0

Post Intermediate Design:

Actual Reactions:

Concentrated Load = 200 lbs.

$R_{PTI.200} := P_{200}$ $R_{PTI.200} = 200.00$ lbs

Uniform Load = 50 plf

$R_{PTI.50} := 1.0 \cdot \left(\frac{q_{50}}{12} \right) \cdot L$ $R_{PTI.50} = 200.00$ lbs

For Wind Pressure

$R_{PTI.wind} := 1.0 \cdot \left(\frac{q_{wind}}{144} \right) \cdot H_{post} \cdot L$ $R_{PTI.wind} = 140.00$ lbs

$R_{PTI} := \max\left(R_{PTI.200}, R_{PTI.50}, R_{PTI.wind}\right)$ $R_{PTI} = 200.00$ lbs

Actual Moment:

$M_{PTI} := R_{PTI} \cdot H_{post}$ $M_{PTI} = 8400.00$ in - lb

Actual Shear:

$V_{PTI} := R_{PTI}$ $V_{PTI} = 200.00$ lbs.

Section Required:

Bending Design: Section Modulus Required:

$S_{PTI.xx} := \frac{M_{PTI}}{Fb_{PT.I}}$ $S_{PTI.xx} = 0.44$ in^3

Shear Design: Area Required:

$A_{PTI.xx} := \frac{1.5 \cdot V_{PTI}}{Fv_{PT.I}}$ $A_{PTI.xx} = 0.03$ in^2

Post Intermediate Section Provided:

$BENDING_{PT.I} := \text{if}\left(S_{PTI.xx} \geq Sx_{PT.I}, \text{"N.G"}, \text{"OK"}\right)$ $BENDING_{PT.I} = \text{"OK"}$

$SHEAR_{PT.I} := \text{if}\left(A_{PTI.xx} \geq A_{PT.I}, \text{"N.G"}, \text{"OK"}\right)$ $SHEAR_{PT.I} = \text{"OK"}$

Post Anchor Along X-X Axis Design:

Actual Reactions:

Concentrated Load = 200 lbs.

$R_{PT.200} := P_{200}$ $\quad R_{PT.200} = 200.00$ lbs

Uniform Load = 50 plf

$R_{PT.50} := 1.0 \cdot \left(\frac{q_{50}}{12} \right) \cdot L$ $\quad R_{PT.50} = 200.00$ lbs

For Wind Pressure

$R_{PT.wind} := 1.0 \cdot \left(\frac{q_{wind}}{144} \right) \cdot H_{post} \cdot L$ $\quad R_{PT.wind} = 140.00$ lbs

$R_{PT.xx} := \max\left(R_{PT.200}, R_{PT.50}, R_{PT.wind}\right)$ $\quad R_{PT.xx} = 200.00$ lbs

Actual Moment:

$M_{PT.xx} := R_{PT.xx} \cdot H_{post}$ $\quad M_{PT.xx} = 8400.00$ in - lb

Actual Shear:

$V_{PT.xx} := R_{PT.xx}$ $\quad V_{PT.xx} = 200.00$ lbs.

Section Required:

Bending Design: Section Modulus Required:

$S_{PT.xx} := \frac{M_{PT.xx}}{Fb_{PT}}$ $\quad S_{PT.xx} = 0.44$ in^3

Shear Design: Area Required:

$A_{PT.xx} := \frac{1.5 \cdot V_{PT.xx}}{Fv_{PT}}$ $\quad A_{PT.xx} = 0.06$ in^2

Post Anchor Section Provided:

$BENDING_{PT.xx} := \text{if}\left(S_{PT.xx} \geq Sx_{PT}, \text{"N.G"}, \text{"OK"}\right)$ $\quad BENDING_{PT.xx} = \text{"OK"}$

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

Post Anchor Along Y-Y Axis Design (Cable Tension):

Actual Moment:

$$M_{PT.yy} := \frac{T_{cable} \cdot \text{floor}\left(\frac{H_{post}}{s_{cable}}\right) \cdot H_{post}}{8}$$

$M_{PT.yy} = 12355.89$ in − lb

Actual Shear:

$$V_{PT.yy} := \frac{T_{cable} \cdot \text{floor}\left(\frac{H_{post}}{s_{cable}}\right)}{2}$$

$V_{PT.yy} = 1176.75$ lbs.

Section Required:

Bending Design: Section Modulus Required:

$$S_{PT.yy} := \frac{M_{PT.yy}}{Fb_{PT}}$$

$S_{PT.yy} = 0.65$ in^3

Shear Design (Area Required)

$$A_{PT.yy} := \frac{1.5 \cdot V_{PT.yy}}{Fv_{PT}}$$

$A_{PT.yy} = 0.35$ in^2

Post Anchor Section Provided:

$BENDING_{PT.yy} := \text{if}\left(S_{PT.yy} \geq Sy_{PT}, \text{"N.G"}, \text{"OK"}\right)$ $BENDING_{PT.yy} = \text{"OK"}$

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

Post Anchor Combined Stress (Bending at Hpost/2)

$$fb_{PT.xx.0.5H} := \frac{M_{PT.xx}}{2Sx_{PT}}$$

$fb_{PT.xx.0.5H} = 4608.29$ psi

$$fb_{PT.yy} := \frac{M_{PT.yy}}{Sy_{PT}}$$

$fb_{PT.yy} = 14312.39$ psi

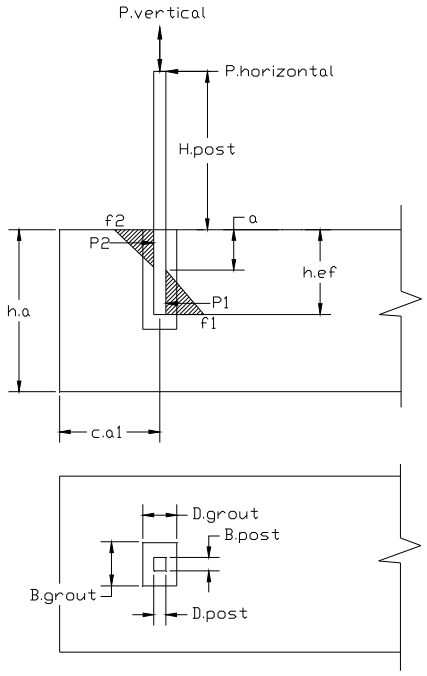
$$IE_{PT} := \frac{fb_{PT.xx.0.5H}}{Fb_{PT}} + \frac{fb_{PT.yy}}{Fb_{PT}}$$

$IE_{PT} = 1.00$

$$COMBINED_{PT} := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if} \quad 1 \geq IE_{PT} \end{vmatrix}$$

$COMBINED_{PT} = \text{"OK"}$

Post Anchorage to Concrete:



Post Data:

$B_{post} := 2.00$ Post Width (in)

$D_{post} := 2.00$ Post Depth (in)

$h_{ef} := 5.00$ Post Embedment (in)

$C_{a1} := 5.50$ Minimum Edge Distance (in)

Grout Size Data:

$B_{grout} := 2.66$ Equivalent Grout Width (in)

$D_{grout} := 2.66$ Equivalent Grout Depth (in)

$f'c_{grout} := 8000.00$ Grout Strength (psi)

Concrete Slab Data:

$f'c_{concrete} := 3000.00$ Concrete Strength (psi)

$d_o := 3.00$ Hold diameter (Grout Diameter) (in)

$h_a := 6.00$ Concrete Slab Deep (in)

Equivalent Actual Horizontal Load at Top Railing Height (Pd)

$Pd := R_{PT.xx}$ $Pd = 200.00$ lbs

Arm Effect:

$$a := \frac{2 \cdot h_{ef}^{\ 2} + 3 \cdot H_{post} \cdot h_{ef}}{3 \cdot h_{ef} + 6.H_{post}}$$ $a = 2.55$ in

$$P_2 := \frac{Pd \cdot a^2}{h_{ef} \cdot (2 \cdot a - h_{ef})}$$ $P_2 = 2770.94$ lbs

$P_1 := P_2 - Pd$ $P_1 = 2570.94$ lbs

$M_{external} := Pd \cdot (H_{post} + a)$ $M_{external} = 8909.36$ lbs – in

$M_{internal} := P_2 \cdot \frac{2}{3} \cdot a + P_1 \cdot \frac{2}{3} \cdot (h_{ef} - a)$ $M_{internal} = 8909.36$ lbs – in

$Vu := 1.6 \cdot (\max(P_1, P_2))$ $Vu = 4433.50$ lbs

Stress Analysis Between Post and Grout:

$$f_{2.grout} := 1.6 \cdot \frac{2 \cdot P_2}{a \cdot B_{post}}$$

$f_{2.grout} = 1740.80$ psi

$$f_{1.grout} := 1.6 \cdot \frac{2.P_1}{(h_{ef} - a) \cdot B_{post}}$$

$f_{1.grout} = 1676.80$ psi

Bearing Stress for Grout :

$$Fp_{grout} := \left[0.65 \cdot \left(0.85 f'c_{grout}\right)\right] \cdot 1.5$$

$Fp_{grout} = 6630.00$ psi

$$Stress_{grout} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} \quad \text{if } \max\left(f_{1.grout}, f_{2.grout}\right) \le Fp_{grout} \end{cases}$$

$Stress_{grout} = \text{"O.K."}$

Stress Analysis Between Grout and Concrete:

$$f_{2.concrete} := 1.6 \frac{2 \cdot P_2}{a \cdot B_{grout}}$$

$f_{2.concrete} = 1308.87$ psi

$$f_{1.concrete} := 1.6 \cdot \frac{2.P_1}{(h_{ef} - a) \cdot B_{grout}}$$

$f_{1.concrete} = 1260.75$ psi

Bearing Stress for Concrete :

$$Fp_{concrete} := \left[0.65 \cdot \left(0.85 \cdot f'c_{concrete}\right)\right] \cdot 1.5$$

$Fp_{concrete} = 2486.25$ psi

$$Stress_{concrete} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} \quad \text{if } \max\left(f_{1.concrete}, f_{2.concrete}\right) \le Fp_{concrete} \end{cases}$$

$Stress_{concrete} = \text{"O.K."}$

Shear Analysis-Concrete Breakout Strength:

$A_{Vco} := 4.5 \cdot C_{a1}^{2}$ $\quad A_{Vco} = 136.13$ in^2

$$A_{Vc} := \begin{cases} 2 \cdot (1.5 \cdot C_{a1}) \cdot h_a & \text{if } h_a < 1.5 \cdot C_{a1} \\ 2 \cdot (1.5 \cdot C_{a1}) \cdot (1.5 \cdot C_{a1}) & \text{if } h_a \geq 1.5 \cdot C_{a1} \end{cases}$$ $\quad A_{Vc} = 99.00$ in^2

$\psi_{ed.V} := 1.00$ Modification factor for edge effect

$\psi_{c.V} := 1.40$ Modification Factor in No cracked concrete at service loads
ψc.v=1.4
Modification Factor in cracked concrete with
ψc.v=1 w/ No supplementary reinforcement.
ψc.v=1.2 w/ Supplementary reinforcement of a No 4 Bar between the anchor and the edge.
ψc.v=1.4 w/ Supplementary reinforcement of No 4 Bar and stirrups spaced at not more than 4 in

$l_e := \text{if}(h_{ef} > 8 \cdot d_o, 8 \cdot d_o, h_{ef})$ $\quad l_e = 5.00$ in

$$V_b := \begin{cases} 7 \cdot \left[\left(\frac{l_e}{d_o} \right)^{0.2} \sqrt{d_o} \cdot \sqrt{f'c_{concrete}} \cdot C_{a1}^{1.5} \right] \\ 9 \cdot \left(\sqrt{f'c_{concrete}} \cdot C_{a1}^{1.5} \right) \end{cases}$$ $\quad V_b = 6358.39$ lbs.

$V_{cb} := \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ed.V} \cdot \psi_{c.V} \cdot V_b$ $\quad V_{cb} = 6474$ lbs.

$\phi := 0.70$ Conditions
ϕ=0.75 w/ Supplementary reinforcement are crossing concrete failure prism.
ϕ=0.70 w/ No supplementary reinforcement are crossing concrete failure prism.

$\phi \cdot V_{cb} = 4531.80$ lbs

$BREAKOUT_{shear} := \text{if}(Vu > \phi \cdot V_{cb}, \text{"N.G."}, \text{"OK"})$ $\quad BREAKOUT_{shear} = \text{"OK"}$

Shear Analysis-Concrete Burst-Out Strength:

$$C_{a1c} := \begin{cases} C_{a1} \\ \text{"N/A"} \quad \text{if } h_a < C_{a1} \end{cases}$$

$C_{a1c} = 5.50$ in

Total Cone Area for Failure Plane

$$A_{cone.total} := \sqrt{2} \cdot C_{a1c} \cdot \pi \cdot \left(C_{a1c} - \frac{D_{grout}}{2} + \frac{B_{grout}}{2} \right)$$

$A_{cone.total} = 134.40$ in^2

Cone Area for Failure Plane at Post

$$A_{cone.post} := \sqrt{2} \cdot \left(\frac{B_{grout}}{2} \right)^2 \cdot \pi$$

$A_{cone.post} = 7.86$ in^2

Final Cone Area for Failure Plane

$$A_{cone.final} := \frac{A_{cone.total} - A_{cone.post}}{2}$$

$A_{cone.final} = 63.27$ in^2

Final Area for Failure Plane

$$V_{cone.cb} := 4 \cdot \sqrt{f'c_{concrete}} \cdot A_{cone.final}$$

$V_{cone.cb} = 13861.56$ lbs

$\phi_{B.O} := 0.75$

Conditions
ϕ=0.75 for Any Condition

$\phi_{B.O} \cdot V_{cone.cb} = 10396.17$ lbs

$$BURSTOUT_{shear} := \text{if}\left(Vu > \phi_{B.O} \cdot V_{cone.cb}, \text{"N.G."}, \text{"OK"} \right)$$

$BURSTOUT_{shear} = \text{"OK"}$