Job: 13125 SW 63rd Ave, Miami.

LOCATION: 13125 SW 63rd Ave, Miami, FL 33156

STRUCTURAL CALCULATIONS

DESIGN CRITERIA:

Calculations based on:

1. 2023 Florida Building Code
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-15 Ed
5. Specifications for the Design of Cold-Formed Stainless Steel Structural Members SEI/ASCE8-02
6. ADM1-2020 Aluminum Design Manual

CALCULATION INDEX:

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.

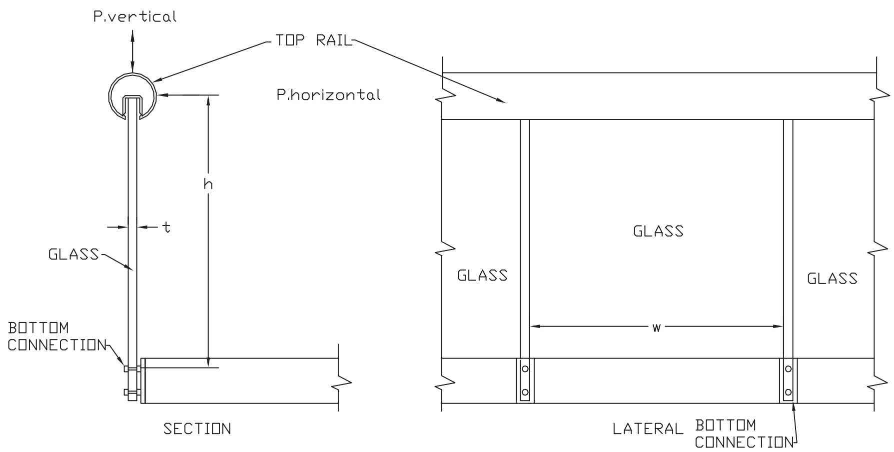
Digitally signed by Alaym Aguilar
Date: 2026.04.10 16:56:37 -04'00'

Alaym Aguilar, PE
Lic. No. 92154

Alaym Aguilar, Professional Engineer, State of Florida, License No. 92154. This item has been digitally signed and sealed by Alaym Aguilar, PE, on the date provided on the signature. Printed copies of this document are not considered signed and sealed and the signature must be verified on any copies.

INTERIOR STEEL STAIR WITH GLASS RAILING DESIGN & ANALYSIS

Glass Railing Design



Loads Data:

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

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

$q_{wind} := 10.00$ Uniform Distributed Load (psf)

Glass Data:

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

$Mr_{flexure} := 24000.00$ Modulus of Rupture of Glass in Flexure (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 Pannel in Cantilever (in)

$t := 0.5315$ Thickness of Glass Pannel (in)

$w := 48.00$ Width of Glass Pannel (in)

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

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

$L := \begin{cases} h \\ w \text{ if } w < h \end{cases}$ $L = 45.00$ in

$S_x := \frac{L \cdot t^2}{6}$ $S_x = 2.12$ in^3

$I_x := \frac{L \cdot t^3}{12}$ $I_x = 0.56$ in^4

$A := L \cdot t$ $A = 23.92$ 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

$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$ $M_{50} = 8437.50$ lbs − in

Uniform Distributed Wind Load

$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$ $M_{wind} = 3164.06$ 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

$V_{50} := q_{50} \cdot \frac{L}{12}$ $V_{50} = 187.50$ lbs

Uniform Distributed Wind Load

$V_{wind} := \frac{q_{wind}}{144} \cdot w \cdot h$ $V_{wind} = 150.00$ lbs

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

Section Required:

Bending Design:

Section Modulus Required

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

Shear Design:

Area Required

$$A_r := \frac{V_{actual}}{F_v} \qquad A_r = 0.07 \quad in^2$$

Section Provided:

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

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

Check Deflection:

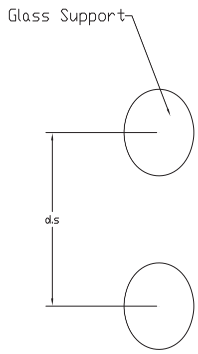
$$\Delta_{allow} := \frac{h}{30} \qquad \Delta_{allow} = 1.50 \quad in$$

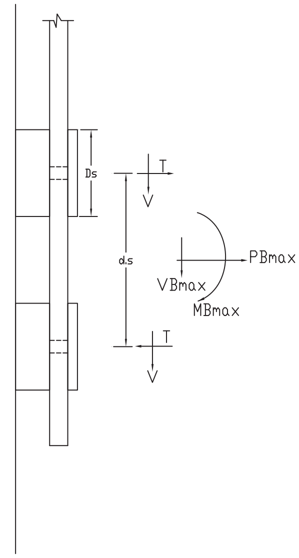
$$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_x} \qquad \Delta_{200} = 1.04 \quad in$$

$$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_x} \qquad \Delta_{wind} = 0.27 \quad in$$

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

Connection Design:





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

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

$P_{system} := 250.00$ Pannel Total Weigth of Glass and Handrail (lbs)

$Standoff_{pairs} := 3.00$ Standoff Pair per panel

Maximum Connection Moment:

$$M_{Bactual} := \frac{M_{actual}}{Standoff_{pairs}} \qquad M_{Bactual} = 3000.00 \quad lbs - in$$

Maximum Connection Shear:

$$V_{Bactual} := \frac{V_{actual}}{Standoff_{pairs}} + \frac{P_{system}}{Standoff_{pairs}} \qquad V_{Bactual} = 150.00 \quad lbs$$

Maximum Connection Axial:

$$P_{Bactual} := \frac{V_{actual}}{Standoff_{pairs}} \qquad P_{Bactual} = 66.67 \quad lbs$$

Tensile Load Per Glass Supports in Connection:

$$T_{bolt} := \frac{M_{Bactual}}{d_s} + P_{Bactual} \qquad T_{bolt} = 816.67 \quad lbs$$

Shear Load Per Glass Supports in Connection:

$$V_{bolt} := \frac{\max(V_{200}, V_{50})}{Standoff_{pairs}} + \frac{P_{system}}{Standoff_{pairs}} \qquad V_{bolt} = 150.00 \quad lbs$$

Check Two Way Glass Shear at Glass Supports in Connection:

$A_p := \pi \cdot (D_s + t)$ — $A_p = 7.95$ in^2

$f_v := \dfrac{T_{bolt}}{A_p \cdot t}$ — $f_v = 193.20$ psi

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

Threaded Rod Analysis:

$d_b := 0.375$ Diameter of Threaded Rod (in)

$n := 16.00$ Threads per in

$A_{rod} := 0.7854 \cdot \left(d_b - \dfrac{0.9743}{n}\right)^2$ — $A_{rod} = 0.077$ in^2

$F_{nv} := 31.50$ Nominal Shear Strees (ksi)

$F_{nt} := 52.50$ Nominal Tensile Strees (ksi)

$Rf := 0.9$ Reduce Factor:
= 0.9 D < 1/2"
= 1.0 D>= 1/2"

$SFv := 3.00$ Safe Factor (shear)

$SFt := 3.00$ Safe Factor (tension)

$F_{vb} := F_{nv} \cdot \dfrac{Rf}{SFv}$ — $F_{vb} = 9.45$ ksi

$F_t := F_{nt} \cdot \dfrac{Rf}{SFt}$ — $F_t = 15.75$ ksi

Check Shear

$f_{vrod} := \dfrac{V_{bolt}}{1000 A_{rod}}$ — $f_{vrod} = 1.94$ ksi

$V_{allowrod} := F_v \cdot A_{rod}$ — $V_{allowrod} = 232.47$ kips

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

Check Tension

$f_{trod} := \dfrac{T_{bolt}}{A_{rod}}$ — $f_{trod} = 10539.03$ ksi

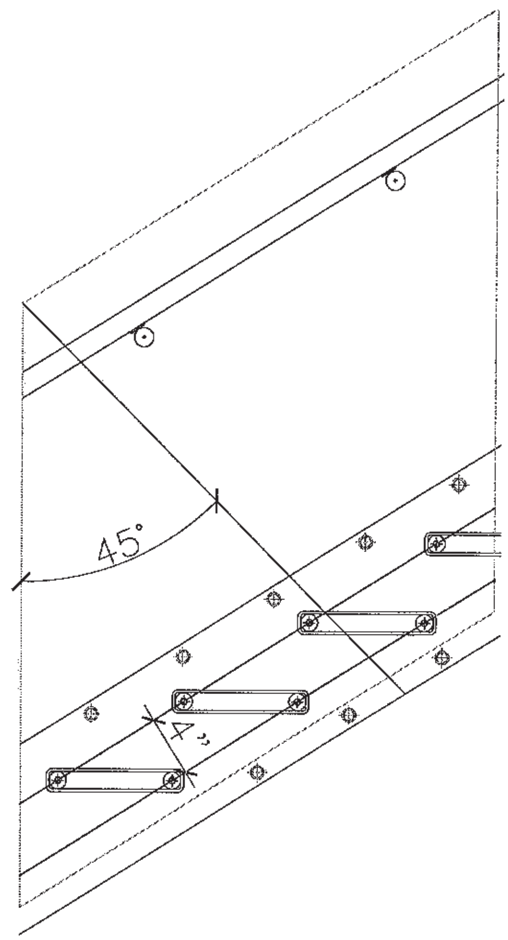
$F'_{nt} := \min\left(F_t, \max\left(0, 1.25F_t - 2.4 \cdot f_{vrod}\right)\right)$ — $F'_{nt} = 15.04$ ksi

$T_{allow} := F'_{nt} \cdot A_{rod} \cdot 1000$ — $T_{allow} = 1165.58$ lbs

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

Minimum Thread Length

$TL_{min} := \dfrac{2 \cdot T_{bolt}}{F_v \cdot \pi \cdot \left(d_b - \dfrac{0.9743}{n}\right)}$ — $TL_{min} = 0.55$ in


45°
4"

S.S Bolt Threaded Tension & Shear Stress Design 14th

Data:

Bolt Type=3/8" Bolt A325

$T_{bolt} := 1.150$ Tension in Anchor (kips) $\Omega := 2.00$ ASD Factor=2.00

$V_{bolt} := 0.2025$ Shear in Anchor (kips)

$F_{nv} := 29.25$ Nominal Shear Strength (ksi) Table 2-4 ....=0.45*Fu

$F_{nt} := 48.75$ Nominal Tensile Strength (ksi) Table 2-4......=0.75*Fu

$D := 0.375$ Screw Diameter (in) $n := 16.00$ Threads per in......... (AISC-Table 7-17 page 7-81)

$F_v := \frac{F_{nv}}{\Omega}$ $F_v = 14.625$ ksi

$F_t := \frac{F_{nt}}{\Omega}$ $F_t = 24.375$ ksi

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

Check Shear

$f_v := \frac{V_{bolt}}{A_{bolt}}$ $f_v = 2.613$ ksi

$V_{allow} := F_v \cdot A_{bolt}$ $V_{allow} = 1.133$ kips

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

Check Tension

$f_t := \frac{T_{bolt}}{A_{bolt}}$ $f_t = 14.841$ ksi

$F'_{nt} := \min\left(F_t, \max\left(0, 1.3F_t - \Omega \cdot \frac{F_t}{F_v} \cdot f_v\right)\right)$ $F'_{nt} = 22.977$ ksi

$T_{allow} := F'_{nt} \cdot A_{bolt}$ $T_{allow} = 1.780$ kips

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

Minimum Thread Length

$TL_{min} := \frac{2 \cdot T_{bolt}}{F_v \cdot \pi \cdot \left(D - \frac{0.9743}{n}\right)}$ $TL_{min} = 0.159$ in

Minimum Base Thickness for Bolt Connection Design AISC 14th

Load Data:

$T_{bolt} := 1.150$ 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:

$T_{table} := 8.64$ Maximum Allowable Tension in Anchor by Table 7-2 page 7-23

$Fu_{table} := 120.00$ Maximun Tensile Strength (ksi) by Table 2-5 page 2-41

$L_{table} := 0.484$ Nut Thread Length (in) For High-Strength Bolts..............Table 7-15 page 7-80
For Non High-Strength Bolts.......Table 7-18 page 7-83

Minimum Thread Length (by Calculations)

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

$TL_{calc} = 0.162$ in

Minimum Thread Length (by Equivalent AISC Standard)

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

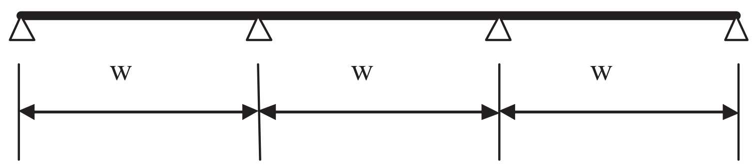
$TL_{equivalent} = 0.133$ in

Minimum Thread Length

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

$TL_{min} = 0.162$ in

Top Railing Design:



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

$F_{b.top} := 16216.00$ Top Railing Allowable Bending Stress (psi)

HANDRAIL WILL BE 1-1/2" O.D. x 1/8" THK. S.S. 304 MIN

$F_{v.top} := 9189.00$ Top Railing Allowable Shear Stress (psi)

$Sx_{top} := 0.17$ Top Railing Inertia Modulus for Vertical Loads (in^3)

$Sy_{top} := 0.17$ Top Railing Inertia Modulus for Horizontal Loads (in^3)

$A_{top} := 0.54$ Top Railing Area (in^2)

Maximum Moment:

Concentrated Load = 200 lbs.

$$M_{200.top} := \frac{P_{200} \cdot w}{5}$$ $M_{200.top} = 1920.00$ lb − in

Uniform Load = 50 plf

$$M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot w^2$$ $M_{50.top} = 971.52$ lb − in

$$M_{max.top} := \max(M_{200.top}, M_{50.top})$$ $M_{max.top} = 1920.00$ lb − in

Maximum Shear:

Concentrated Load = 200 lbs.

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

Uniform Load = 50 plf

$$V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot w$$ $V_{50.top} = 120.00$ lbs

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

Section Required:

Bending Design:

Section Modulus Required

$$Shr := \frac{M_{max.top}}{F_{b.top}}$$ $Shr = 0.12$ in^3

Shear Design:

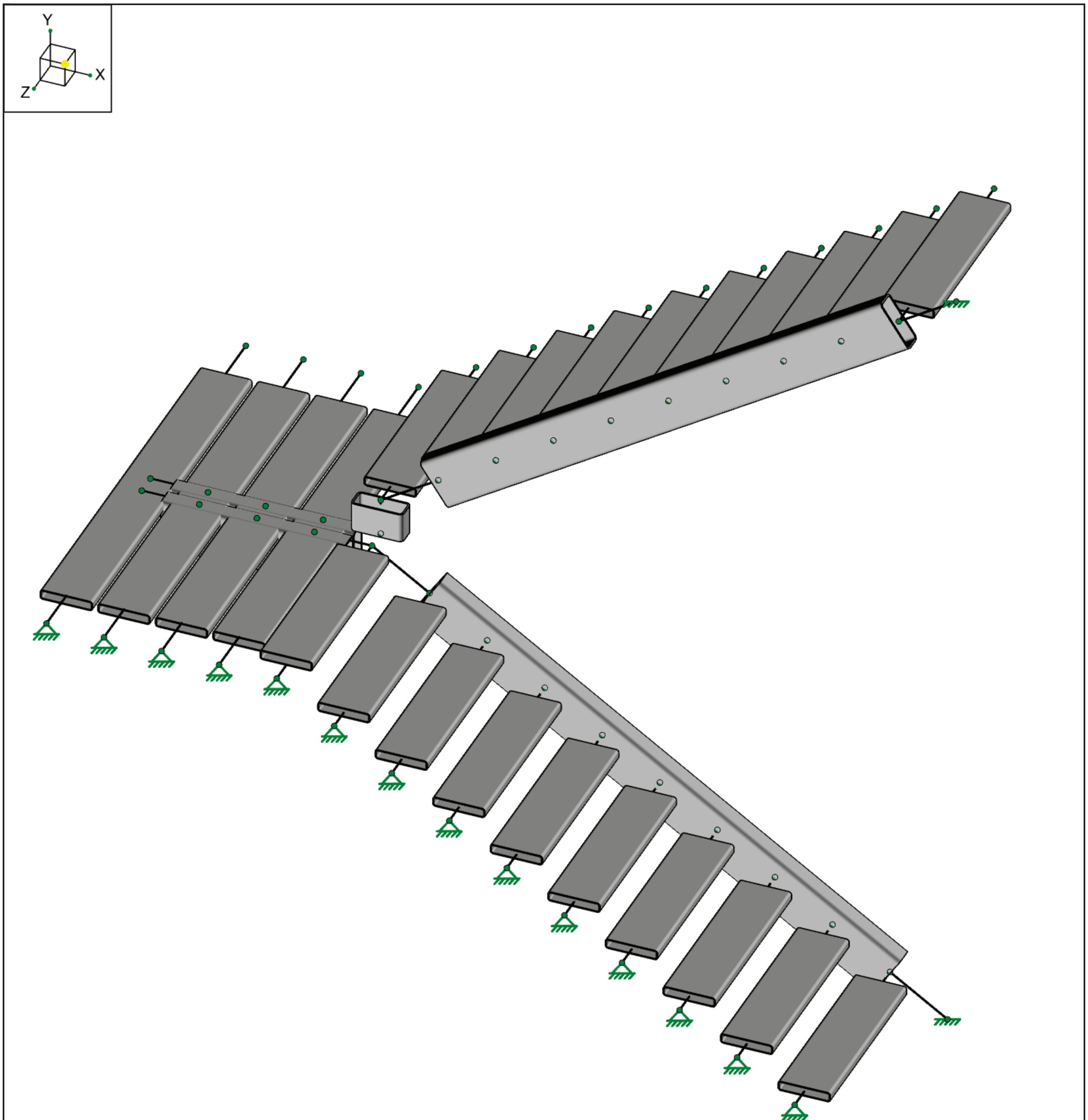
Area Required

$$Ahr := \frac{1.5 \cdot V_{max.top}}{F_{v.top}}$$ $Ahr = 0.03$ in^2

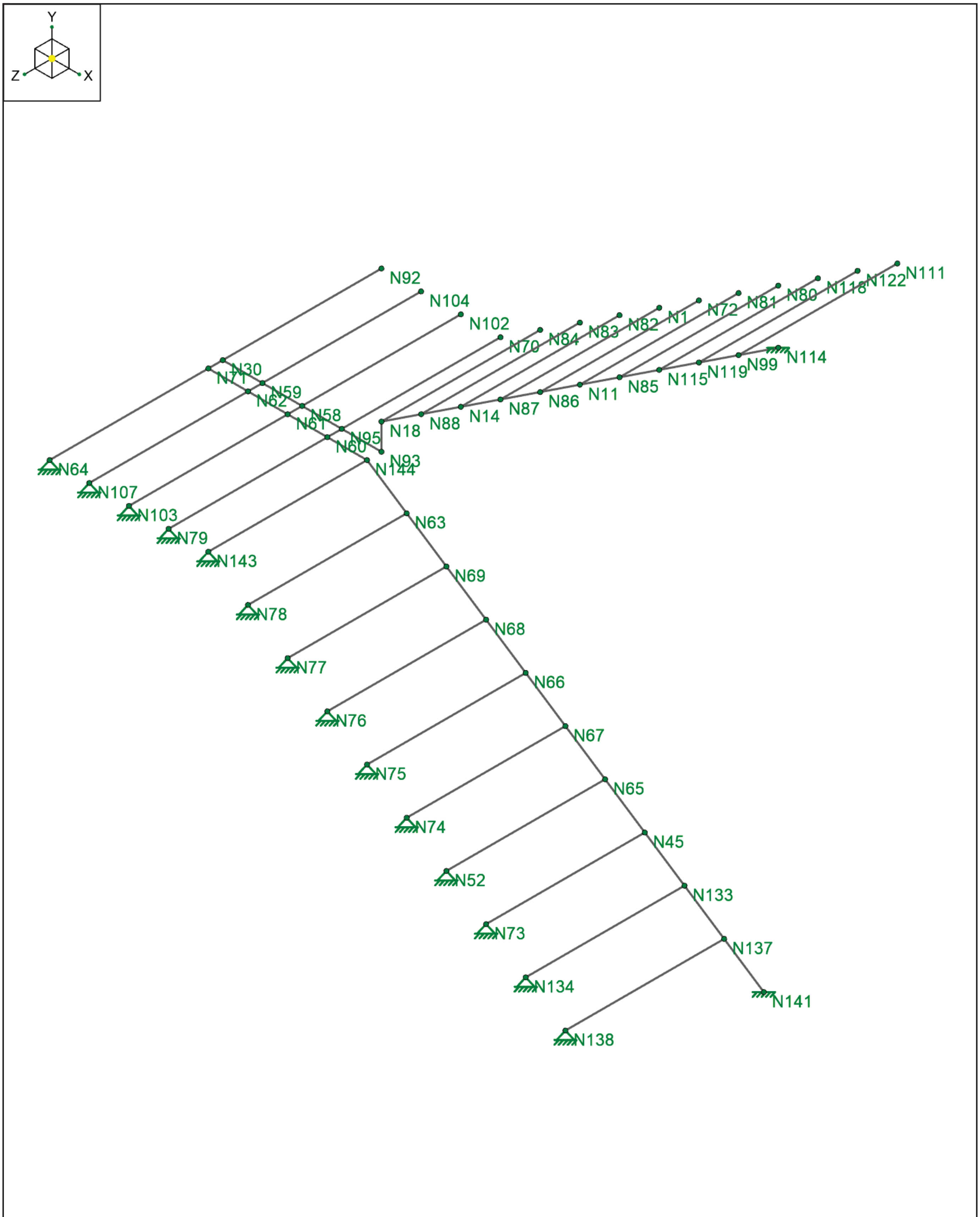
Section Provided:

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

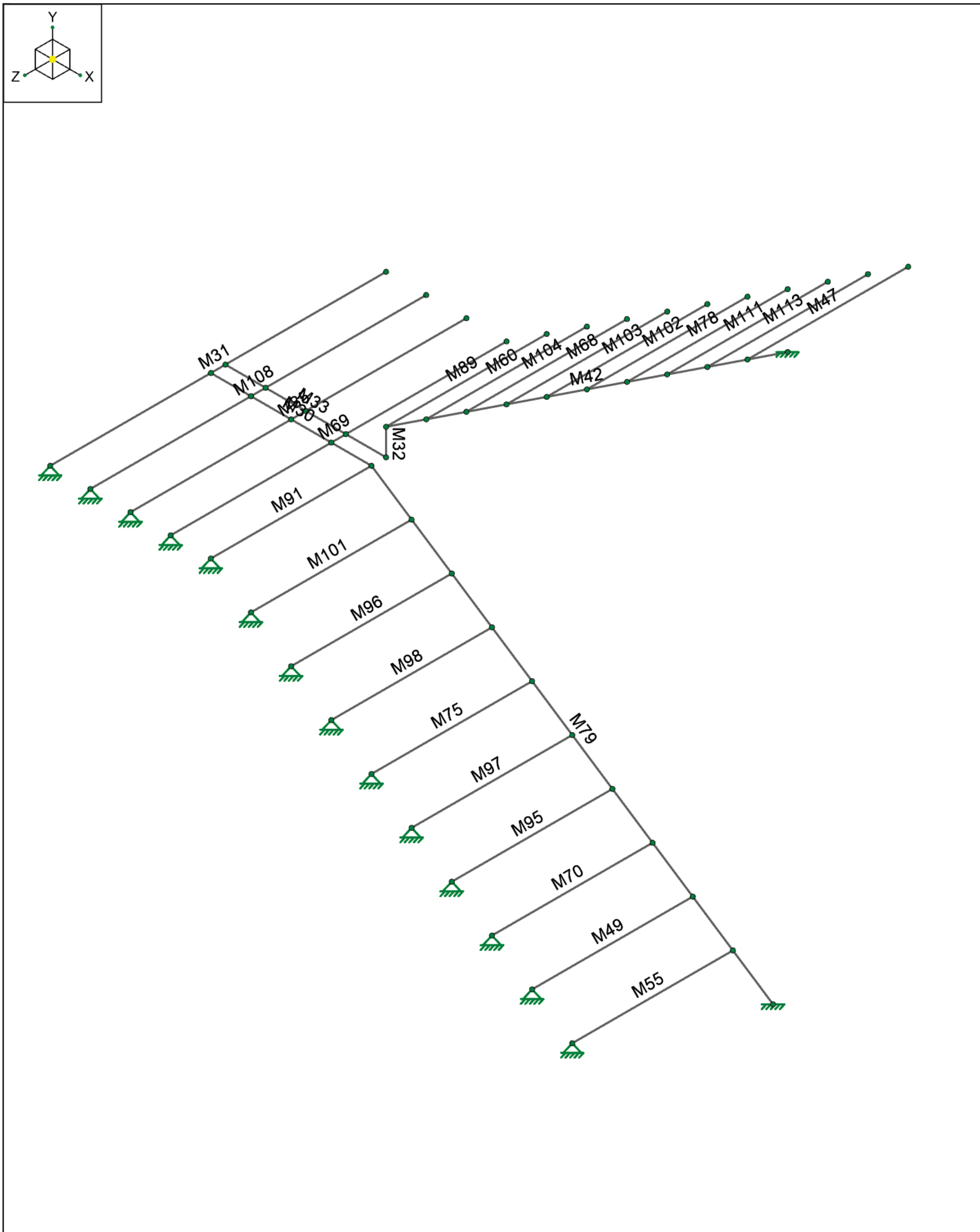
$$SHEAR_{top} := if(Ahr \geq A_{top}, "N.G", "OK")$$ $SHEAR_{top} = "OK"$



RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-16
			25-055 Stair V02.r3d



RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-13
			25-055 Stair V02.r3d



RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-14
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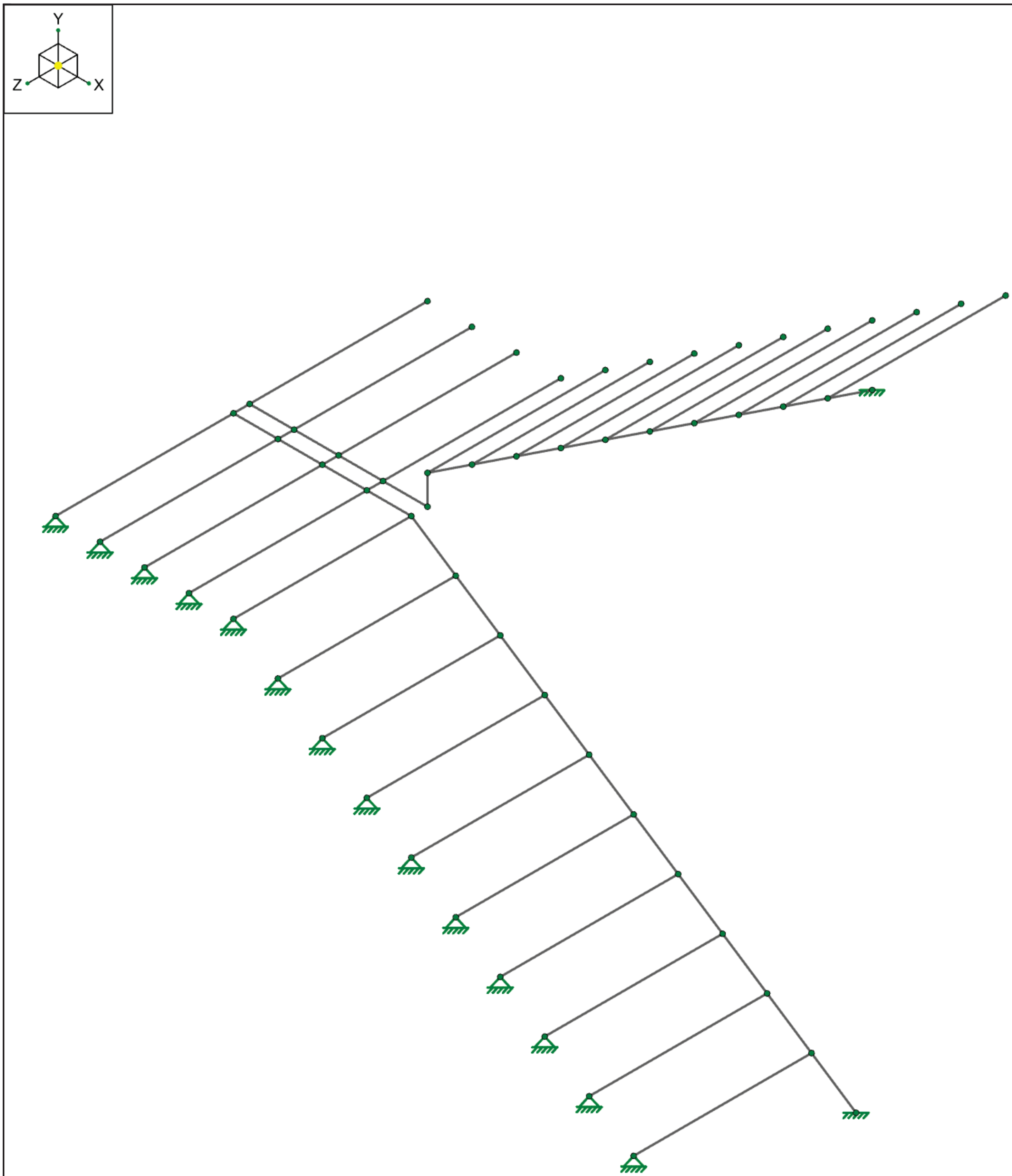
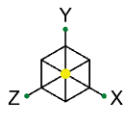
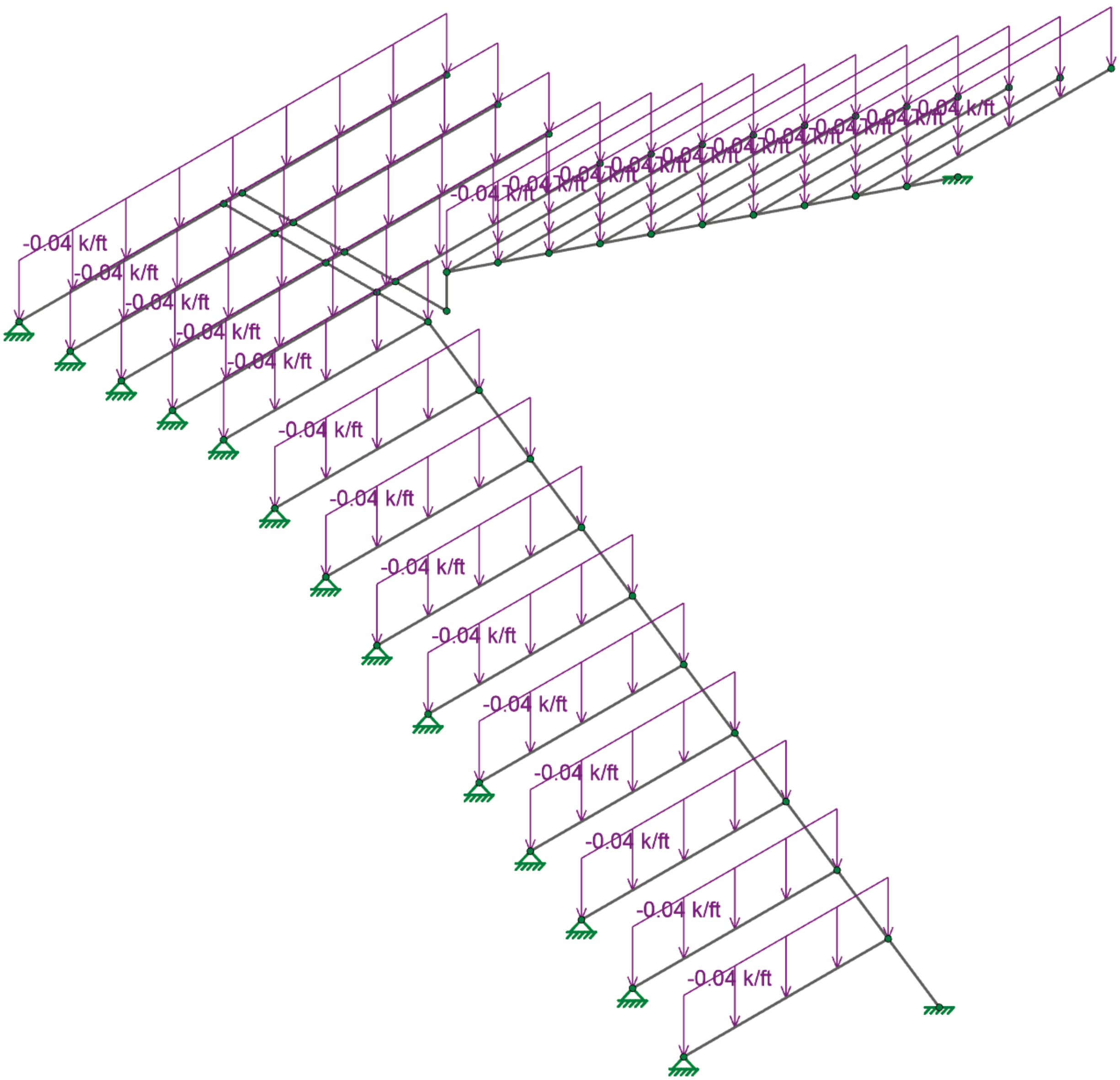


Loads: BLC 1, DL

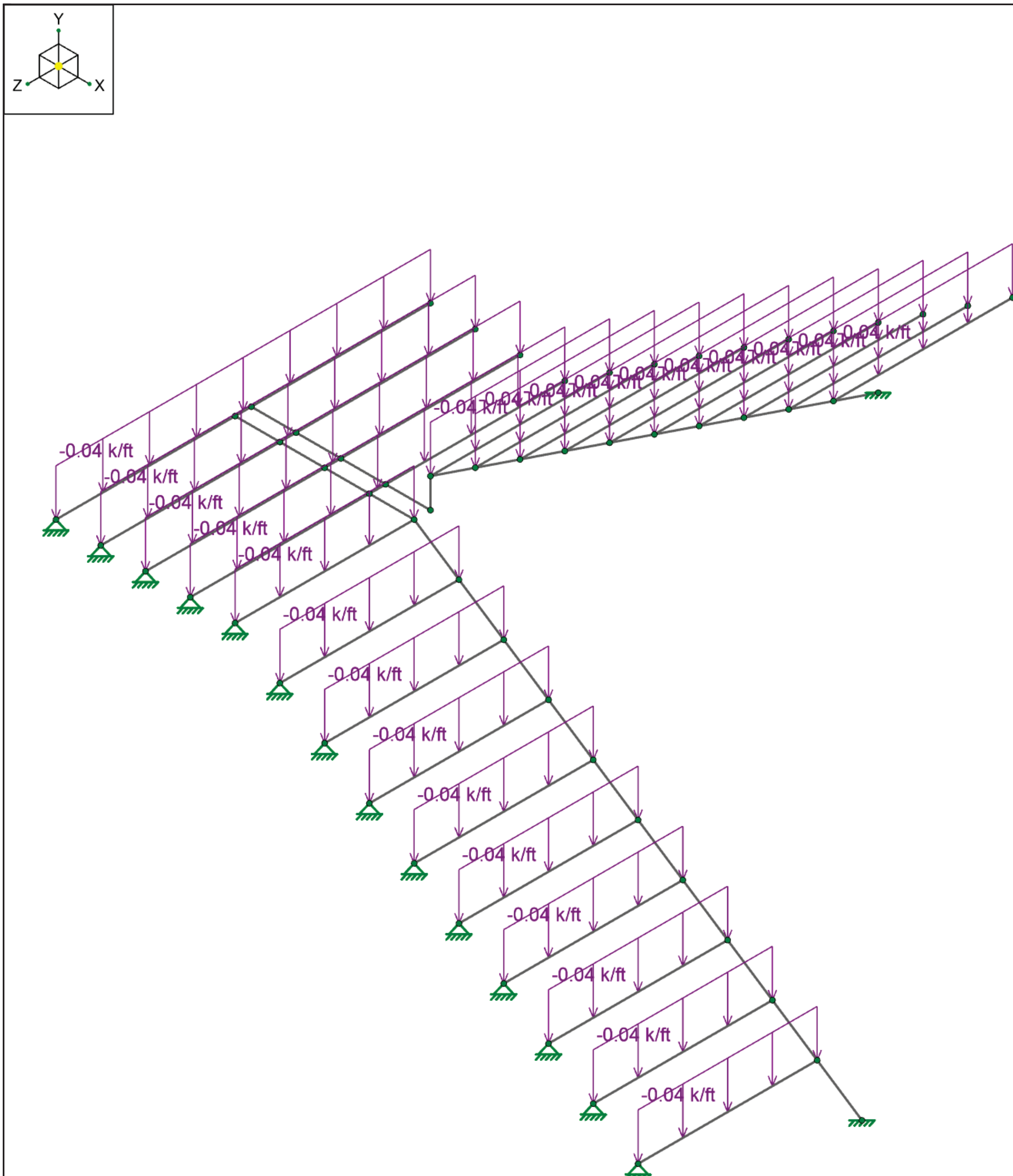
RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-17
			25-055 Stair V02.r3d





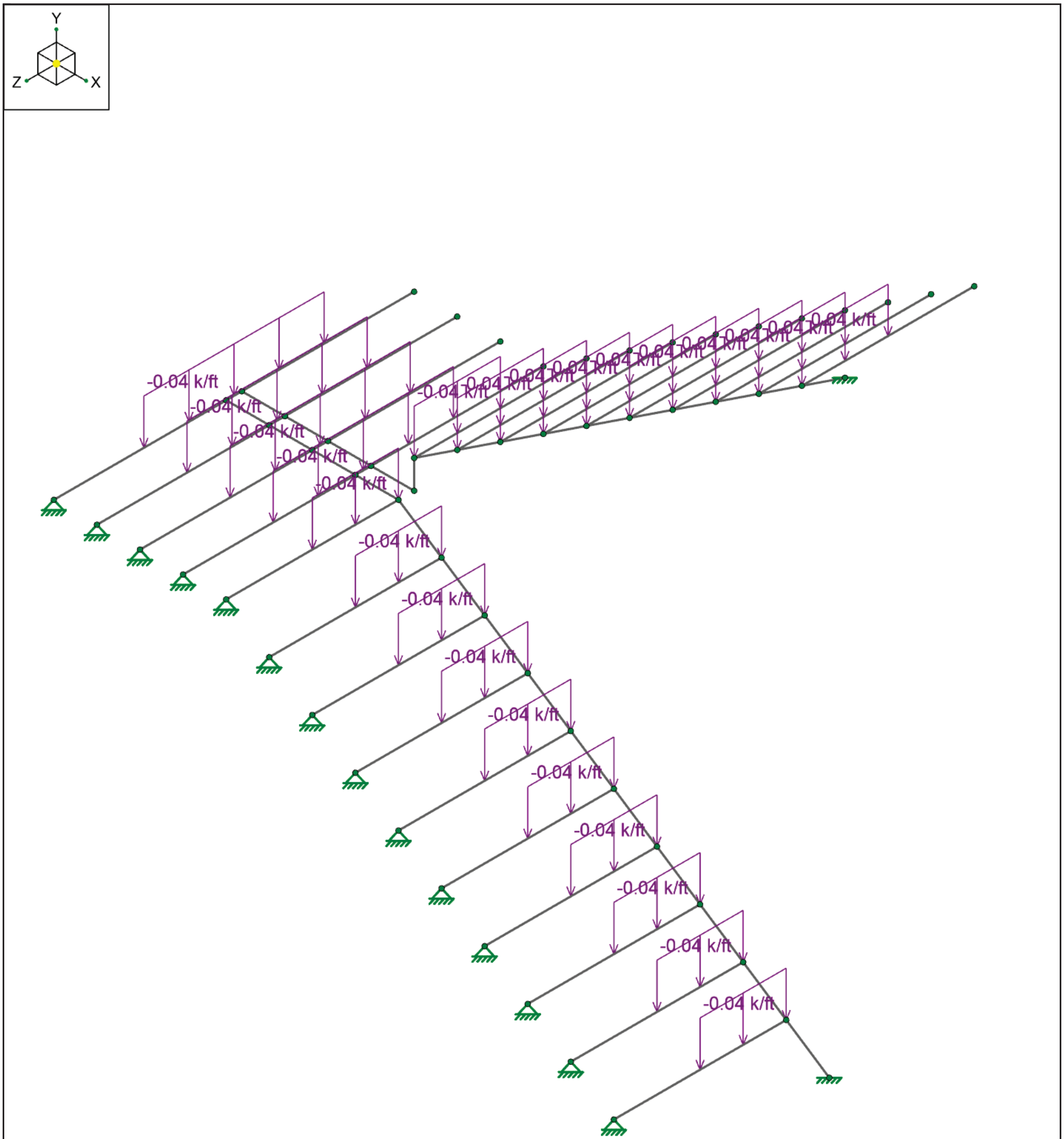
Loads: BLC 2, SI

RISA	3A Engineering Group.		SK-18
			25-055 Stair V02.r3d



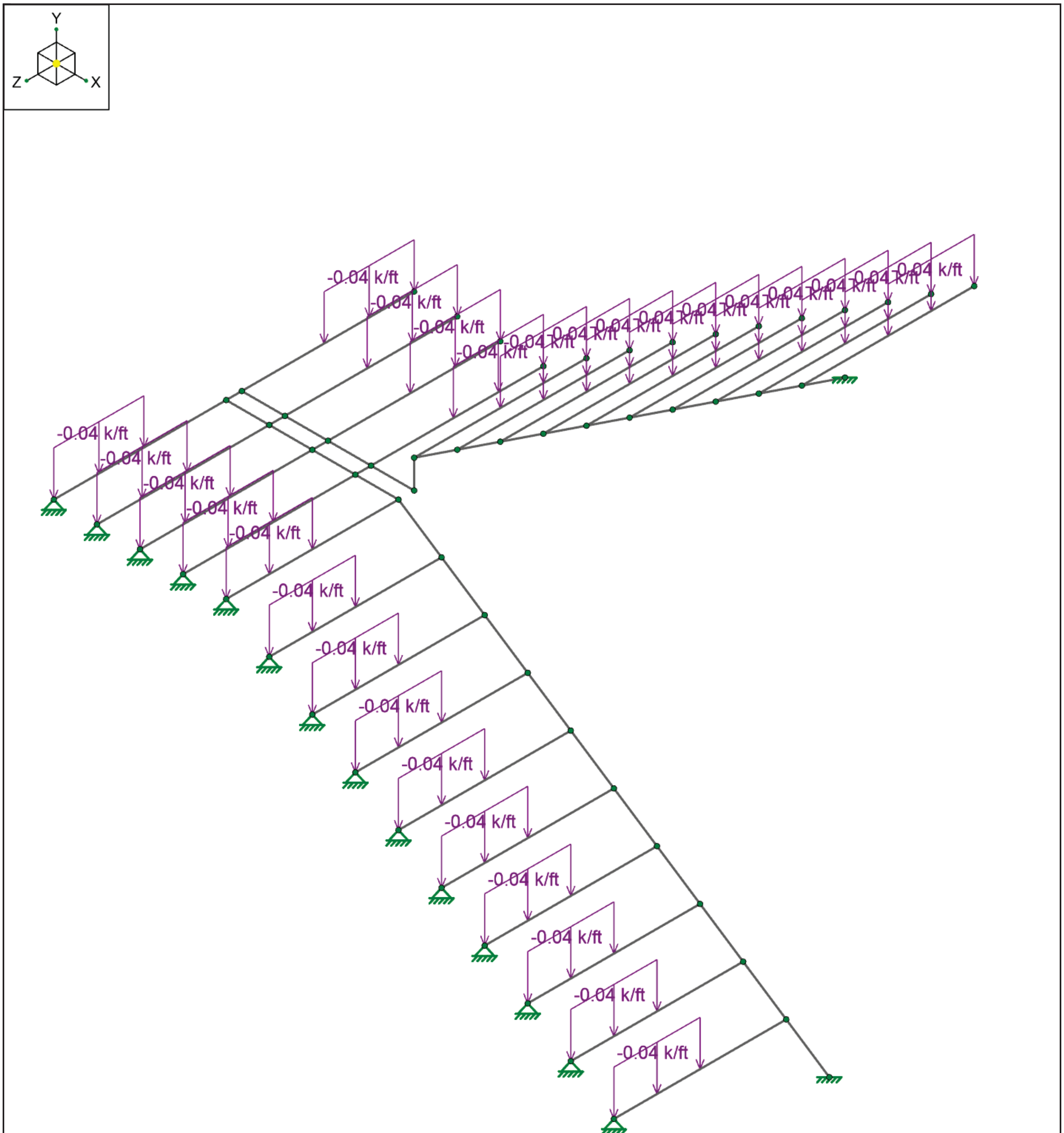
Loads: BLC 3, LL

RISA	3A Engineering Group.		SK-19
			25-055 Stair V02.r3d



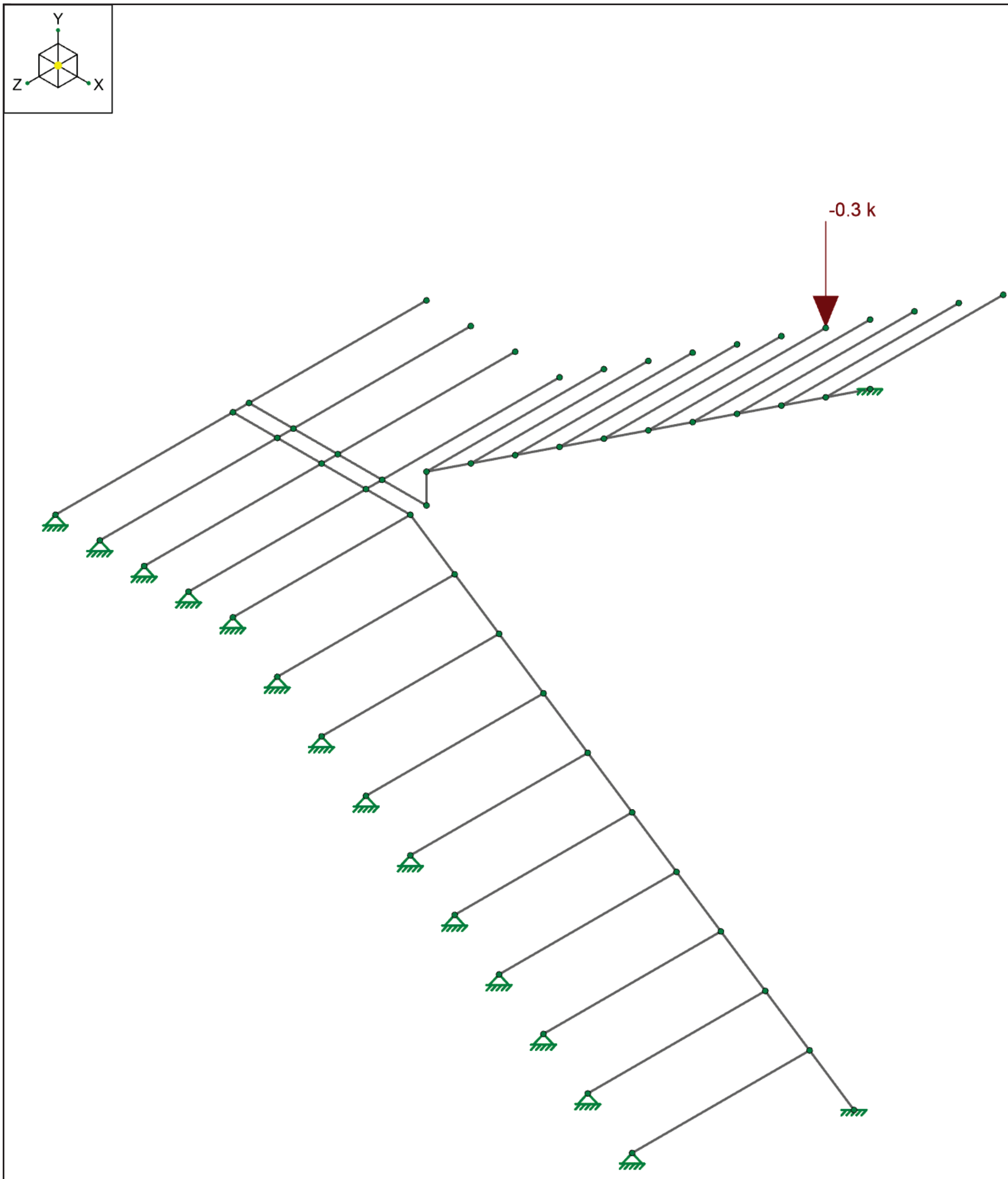
Loads: BLC 4, 0.5LL IN

RISA	3A Engineering Group.		SK-20
			25-055 Stair V02.r3d



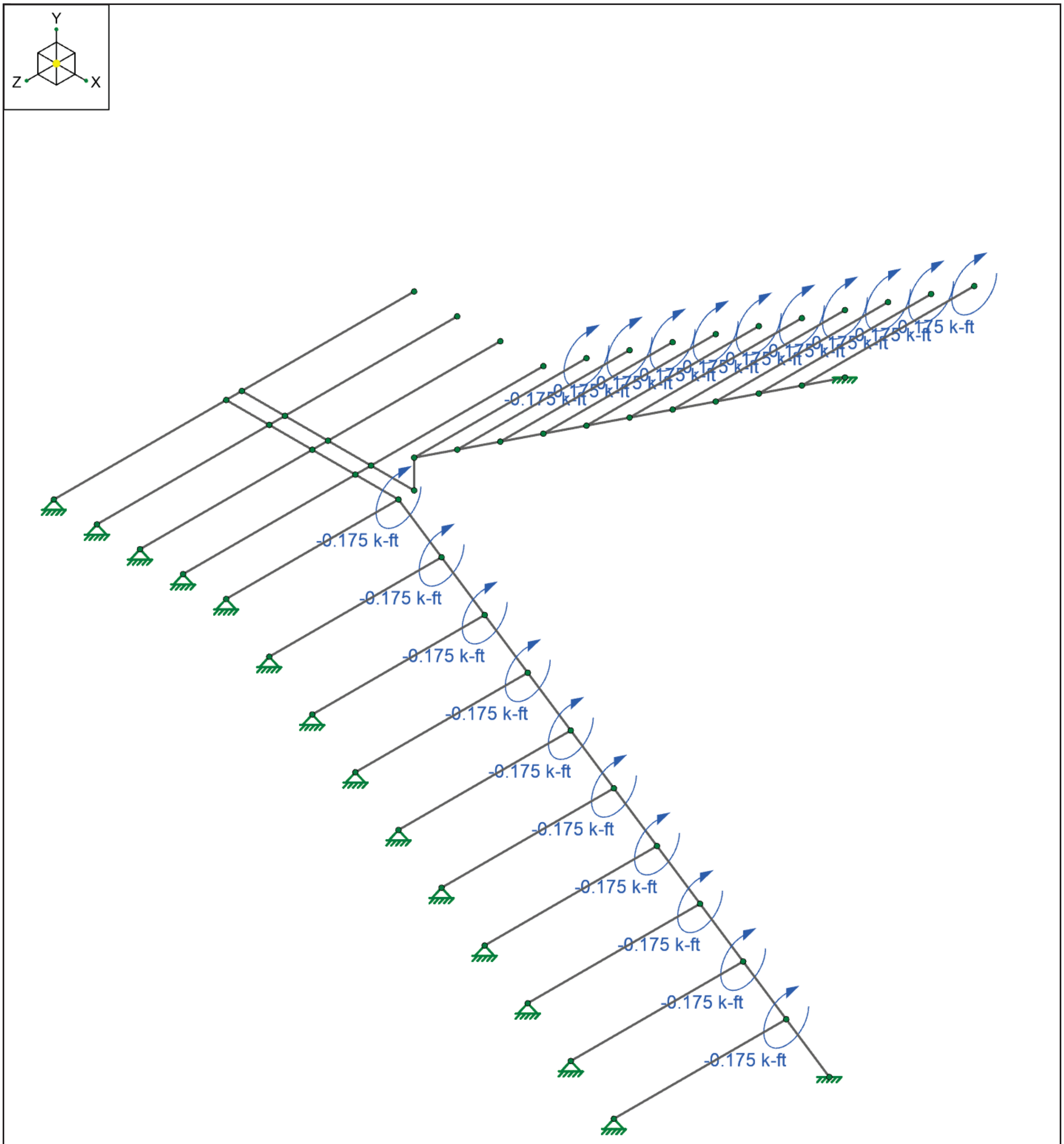
Loads: BLC 7, 0.5LL OUT

RISA	3A Engineering Group.		SK-21
			25-055 Stair V02.r3d



Loads: BLC 5, PL

RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-22
			25-055 Stair V02.r3d



Loads: BLC 6, RAILING

RISA A NEMETSCHEK COMPANY	3A Engineering Group.		SK-23
			25-055 Stair V02.r3d

Node Coordinates

	Label	X [in]	Y [in]	Z [in]	Detach From Diaphragm
1	N1	-44	92.6431	-173	
2	N11	-22	107.1885	-129	
3	N14	-55	85.3704	-129	
4	N18	-77	70.825	-129	
5	N30	-121	63.576525	-129	
6	N45	0	12.6675	-125	
7	N52	-11	19.9402	-81	
8	N63	-66	56.3037	-125	
9	N65	-11	19.9402	-125	
10	N66	-33	34.4856	-125	
11	N67	-22	27.2129	-125	
12	N68	-44	41.7583	-125	
13	N69	-55	49.031	-125	
14	N70	-88	63.5764	-173	
15	N72	-33	99.9158	-173	
16	N73	0	12.6675	-81	
17	N74	-22	27.2129	-81	
18	N75	-33	34.4856	-81	
19	N76	-44	41.7583	-81	
20	N77	-55	49.031	-81	
21	N78	-66	56.3037	-81	
22	N79	-88	63.5764	-81	
23	N80	-11	114.4612	-173	
24	N81	-22	107.1885	-173	
25	N82	-55	85.3704	-173	
26	N83	-66	78.0977	-173	
27	N84	-77	70.825	-173	
28	N85	-11	114.4612	-129	
29	N86	-33	99.9158	-129	
30	N87	-44	92.6431	-129	
31	N88	-66	78.0977	-129	
32	N102	-99	63.5765	-173	
33	N103	-99	63.5765	-81	
34	N104	-110	63.5765	-173	
35	N107	-110	63.5765	-81	
36	N115	0	121.7339	-129	
37	N118	0	121.7339	-173	
38	N119	11	129.0066	-129	
39	N122	11	129.0066	-173	
40	N99	22	136.2793	-129	
41	N111	22	136.2793	-173	
42	N114	33	143.552	-129	
43	N133	11	5.394772	-125	
44	N134	11	5.394772	-81	
45	N137	22	-1.877956	-125	
46	N138	22	-1.877956	-81	
47	N141	33	-9.150684	-125	
48	N143	-77	63.5764	-81	
49	N144	-77	63.5764	-125	
50	N71	-121	63.5765	-125	
51	N58	-99	63.5765	-129	
52	N59	-110	63.5765	-129	
53	N60	-88	63.576425	-125	
54	N61	-99	63.57645	-125	
55	N62	-110	63.576475	-125	

Node Coordinates (Continued)

	Label	X [in]	Y [in]	Z [in]	Detach From Diaphragm
56	N64	-121	63.576525	-81	
57	N92	-121	63.576525	-173	
58	N93	-77	63.5764	-129	
59	N95	-88	63.576431	-129	

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	N138	Reaction	Reaction	Reaction			
2	N134	Reaction	Reaction	Reaction			
3	N73	Reaction	Reaction	Reaction			
4	N52	Reaction	Reaction	Reaction			
5	N74	Reaction	Reaction	Reaction			
6	N75	Reaction	Reaction	Reaction			
7	N76	Reaction	Reaction	Reaction			
8	N77	Reaction	Reaction	Reaction			
9	N78	Reaction	Reaction	Reaction			
10	N143	Reaction	Reaction	Reaction			
11	N79	Reaction	Reaction	Reaction			
12	N103	Reaction	Reaction	Reaction			
13	N107	Reaction	Reaction	Reaction			
14	N64	Reaction	Reaction	Reaction			
15	N141	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
16	N114	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

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

	Member	Shape	Code Check	Loc[in]	LC	Shear Check	Loc[in]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M60	HSS10X2X4	0.1	0	3	0.033	0	z	2	176.967	216.936	11.824	49.55	1	H1-1b
2	M68	HSS10X2X4	0.1	0	3	0.033	0	z	2	176.967	216.936	11.824	49.55	1	H1-1b
3	M69	HSS10X2X4	0.269	44.083	3	0.064	47.917	y	2	96.224	216.936	11.824	49.68	2.02	H1-1b
4	M70	HSS10X2X4	0.019	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
5	M75	HSS10X2X4	0.022	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
6	M78	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
7	M85	HSS10X2X4	0.139	44.083	3	0.032	48.875	z	2	96.224	216.936	11.824	49.68	2.196	H1-1b
8	M89	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
9	M95	HSS10X2X4	0.018	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
10	M96	HSS10X2X4	0.041	44	3	0.021	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
11	M97	HSS10X2X4	0.019	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
12	M98	HSS10X2X4	0.029	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
13	M101	HSS10X2X4	0.06	44	3	0.023	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
14	M102	HSS10X2X4	0.189	0	6	0.049	0	z	6	176.967	216.936	11.824	49.55	1	H1-1b
15	M103	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
16	M104	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
17	M108	HSS10X2X4	0.104	44.083	3	0.032	48.875	z	2	96.224	216.936	11.824	49.68	2.437	H1-1b
18	M111	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
19	M113	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
20	M47	HSS10X2X4	0.1	0	3	0.033	0	z	3	176.967	216.936	11.824	49.55	1	H1-1b
21	M49	HSS10X2X4	0.02	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
22	M55	HSS10X2X4	0.02	44	3	0.02	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
23	M91	HSS10X2X4	0.067	44	3	0.029	44	z	3	176.967	216.936	11.824	49.68	1.667	H1-1b
24	M79	HSS10X4X6	0.114	131.868	3	0.083	131.868	z	3	241.139	371.358	48.3	93.15	3	H1-1b
25	M42	HSS10X4X6	0.102	131.868	3	0.136	131.868	y	3	241.139	371.358	48.3	93.15	2.306	H1-1b
26	M30	HSS10X4X6	0.064	33	3	0.079	44	z	3	353.928	371.358	48.3	93.15	1.331	H1-1b
27	M31	HSS10X2X4	0.092	44.083	3	0.032	48.875	z	2	96.224	216.936	11.824	49.68	2.553	H1-1b

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

	Member	Shape	Code Check	Loc[in]	LC	Shear Check	Loc[in]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
28	M32	HSS10X4X6	0.104	7.249	3	0.1	7.249	y	3	370.874	371.358	48.3	93.15	1.504	H1-1b
29	M33	HSS10X4X6	0.153	44	3	0.053	44	z	3	353.928	371.358	48.3	93.15	2.029	H1-1b

Envelope Member Section Deflections - Strength

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
1	M60	1	max	0.024	3	0.009	3	0.023	3	0	3	NC	6	NC	6
2			min	0.011	6	0.006	1	0.015	6	0	1	NC	1	NC	1
3		2	max	0.024	3	0.03	3	0.072	3	0	3	4296.187	1	1838.383	1
4			min	0.011	6	0.016	1	0.039	1	0	1	2089.131	3	899.295	3
5		3	max	0.024	3	0.051	3	0.135	3	0	3	2148.093	1	828.342	6
6			min	0.011	6	0.026	1	0.068	6	0	1	1044.566	3	394.631	3
7		4	max	0.024	3	0.073	3	0.206	3	0	3	1432.062	1	523.006	6
8			min	0.011	6	0.037	1	0.099	6	0	1	696.377	3	240.606	3
9		5	max	0.024	3	0.094	3	0.283	3	0	3	1074.047	1	380.183	6
10			min	0.011	6	0.047	1	0.13	6	0	1	522.283	3	169.133	3
11	M68	1	max	0.027	3	0.013	3	0.028	3	0	6	NC	6	NC	6
12			min	0.01	6	0.009	1	0.018	1	0	5	NC	1	NC	1
13		2	max	0.027	3	0.042	3	0.079	3	0	6	3159.935	1	1774.837	1
14			min	0.01	6	0.023	1	0.042	1	0	5	1537.137	3	853.145	3
15		3	max	0.027	3	0.071	3	0.144	3	0	6	1579.968	1	799.662	1
16			min	0.01	6	0.036	1	0.073	1	0	5	768.568	3	376.744	3
17		4	max	0.027	3	0.099	3	0.218	3	0	6	1053.312	1	501.546	1
18			min	0.01	6	0.05	1	0.105	1	0	5	512.379	3	230.594	3
19		5	max	0.027	3	0.128	3	0.298	3	0	6	789.984	1	363.256	1
20			min	0.01	6	0.064	1	0.139	1	0	5	384.284	3	162.519	3
21	M69	1	max	0	5	0	2	0	6	0	6	NC	6	NC	6
22			min	0	2	0	6	0	1	0	2	NC	1	NC	1
23		2	max	0	6	0	1	0.003	2	0	6	NC	6	NC	6
24			min	0	2	-0.001	4	-0.001	5	0	2	NC	1	NC	1
25		3	max	0	6	0.001	2	0.016	3	0	6	NC	6	NC	6
26			min	0	3	0	4	0.009	6	0	2	NC	1	5913.793	3
27		4	max	0	6	0.007	3	0.094	3	0	6	NC	6	1848.703	6
28			min	-0.001	3	0.004	1	0.05	6	0	2	NC	1	977.039	3
29		5	max	0	6	0.013	3	0.195	3	0	6	NC	6	902.867	6
30			min	-0.001	3	0.007	1	0.102	6	0	2	7059.767	3	472.673	3
31	M70	1	max	0	6	0	6	0	6	0	6	NC	6	NC	6
32			min	0	1	0	1	0	1	0	4	NC	1	NC	1
33		2	max	0	2	0	5	0.004	2	0	6	NC	6	NC	6
34			min	0	5	0	4	0.002	6	0	4	NC	1	NC	1
35		3	max	0	2	0	6	0.006	2	0	6	NC	6	NC	6
36			min	0	5	-0.001	4	0.003	6	0	4	NC	1	7399.063	2
37		4	max	0	2	-0.001	6	0.006	3	0	6	NC	6	NC	6
38			min	0	5	-0.001	4	0.003	6	0	4	NC	1	7838.346	3
39		5	max	0	2	-0.001	6	0.005	3	0	6	NC	6	NC	6
40			min	0	5	-0.002	4	0.003	6	0	4	NC	1	8124.421	3
41	M75	1	max	0	6	0	6	0	6	0	3	NC	6	NC	6
42			min	0	1	0	1	0	1	0	6	NC	1	NC	1
43		2	max	0	6	0	5	0.005	2	0	3	NC	6	NC	6
44			min	0	3	0	4	0.003	6	0	6	NC	1	9583.916	2
45		3	max	0	6	0	5	0.007	2	0	3	NC	6	NC	6
46			min	0	3	-0.001	4	0.004	6	0	6	NC	1	6222.911	2
47		4	max	0	6	0	2	0.008	3	0	3	NC	6	9606.1	6
48			min	0	3	-0.002	4	0.005	6	0	6	NC	1	5740.525	3
49		5	max	0	6	-0.001	2	0.008	3	0	3	NC	6	NC	6
50			min	0	3	-0.002	4	0.004	6	0	6	NC	1	5421.805	3

Envelope Member Section Deflections - Strength (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
51	M78	1	max	0.015	3	0.01	3	0.02	3	0	1	NC	6	NC	6
52			min	0.008	1	0.007	1	0.013	1	0	3	NC	1	NC	1
53		2	max	0.015	3	0.037	3	0.063	3	0	1	3369.563	1	2146.41	1
54			min	0.008	1	0.02	1	0.033	1	0	3	1639.744	3	1015.535	3
55		3	max	0.015	3	0.064	3	0.12	3	0	1	1684.781	1	947.461	1
56			min	0.008	1	0.033	1	0.059	1	0	3	819.872	3	438.701	3
57		4	max	0.015	3	0.091	3	0.186	3	0	1	1123.188	1	587.813	1
58			min	0.008	1	0.046	1	0.088	1	0	3	546.581	3	264.947	3
59		5	max	0.015	3	0.118	3	0.258	3	0	1	842.391	1	423.239	1
60			min	0.008	1	0.059	1	0.117	1	0	3	409.936	3	185.069	3
61	M85	1	max	0	5	0	2	0	6	0	6	NC	6	NC	6
62			min	0	2	0	6	0	1	0	2	NC	1	NC	1
63		2	max	0	5	0	2	0.004	2	0	6	NC	6	NC	6
64			min	0	1	0	4	-0.001	5	0	2	NC	1	NC	1
65		3	max	0	5	0.001	5	0.02	2	0	6	NC	6	8565.892	6
66			min	0	1	0	4	0.011	6	0	2	NC	1	4715.844	2
67		4	max	0	5	0.002	3	0.093	3	0	6	NC	6	1883.215	6
68			min	0	1	0.001	1	0.049	6	0	2	NC	1	987.405	3
69		5	max	0	5	0.003	3	0.189	3	0	6	NC	6	933.965	6
70			min	0	1	0.002	1	0.099	6	0	2	NC	1	486.721	3
71	M89	1	max	0.018	3	0.006	3	0.02	3	0	3	NC	6	NC	6
72			min	0.009	6	0.004	1	0.012	6	0	1	NC	1	NC	1
73		2	max	0.018	3	0.021	3	0.065	3	0	3	6181.607	1	2003.475	6
74			min	0.009	6	0.011	1	0.034	6	0	1	3005.557	3	974.118	3
75		3	max	0.018	3	0.036	3	0.124	3	0	3	3090.804	1	905.581	6
76			min	0.009	6	0.018	1	0.06	6	0	1	1502.779	3	423.157	3
77		4	max	0.018	3	0.05	3	0.191	3	0	3	2060.536	1	568.966	6
78			min	0.009	6	0.025	1	0.089	6	0	1	1001.852	3	256.414	3
79		5	max	0.018	3	0.065	3	0.265	3	0	3	1545.402	1	412.477	6
80			min	0.009	6	0.033	1	0.119	6	0	1	751.389	3	179.506	3
81	M95	1	max	0	6	0	6	0	6	0	6	NC	6	NC	6
82			min	0	1	0	1	0	1	0	4	NC	1	NC	1
83		2	max	0	6	0	5	0.004	2	0	6	NC	6	NC	6
84			min	0	3	-0.001	4	0.003	6	0	4	NC	1	9924.045	2
85		3	max	0	6	-0.001	6	0.007	2	0	6	NC	6	NC	6
86			min	0	3	-0.001	4	0.004	6	0	4	NC	1	6527.023	2
87		4	max	0	6	-0.001	6	0.007	3	0	6	NC	6	NC	6
88			min	0	3	-0.002	4	0.004	6	0	4	NC	1	6306.794	3
89		5	max	0	6	-0.001	6	0.007	3	0	6	NC	6	NC	6
90			min	0	3	-0.003	4	0.004	6	0	4	NC	1	6216.894	3
91	M96	1	max	0	6	0	6	0	6	0	4	NC	6	NC	6
92			min	0	1	0	1	0	1	0	2	NC	1	NC	1
93		2	max	0	6	0.001	5	0.004	2	0	4	NC	6	NC	6
94			min	0	3	0	6	0.002	6	0	2	NC	1	NC	1
95		3	max	0	6	0.001	5	0.006	2	0	4	NC	6	NC	6
96			min	0	3	0	6	0.004	6	0	2	NC	1	7423.832	2
97		4	max	0	6	0.001	2	0.006	2	0	4	NC	6	NC	6
98			min	0	3	0	4	0.004	6	0	2	NC	1	7377.921	2
99		5	max	0	6	0.001	2	0.007	3	0	4	NC	6	NC	6
100			min	0	3	-0.001	4	0.004	6	0	2	NC	1	6269.794	3
101	M97	1	max	0	6	0	6	0	6	0	2	NC	6	NC	6
102			min	0	1	0	1	0	1	0	4	NC	1	NC	1
103		2	max	0	6	0	5	0.005	2	0	2	NC	6	NC	6
104			min	0	3	-0.001	4	0.003	6	0	4	NC	1	9500.294	2
105		3	max	0	6	-0.001	6	0.007	3	0	2	NC	6	NC	6

Envelope Member Section Deflections - Strength (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
106			min	0	3	-0.001	4	0.004	6	0	4	NC	1	6172.986	3
107		4	max	0	6	-0.001	6	0.008	3	0	2	NC	6	9757.728	6
108			min	0	3	-0.002	4	0.005	6	0	4	NC	1	5724.079	3
109		5	max	0	6	-0.001	1	0.008	3	0	2	NC	6	NC	6
110			min	0	3	-0.003	4	0.004	6	0	4	NC	1	5504.57	3
111	M98	1	max	0	6	0	6	0	6	0	3	NC	6	NC	6
112			min	0	1	0	1	0	1	0	6	NC	1	NC	1
113		2	max	0	6	0	5	0.004	2	0	3	NC	6	NC	6
114			min	0	3	0	4	0.003	6	0	6	NC	1	NC	1
115		3	max	0	6	0	2	0.007	2	0	3	NC	6	NC	6
116			min	0	3	0	4	0.004	6	0	6	NC	1	6632.443	2
117		4	max	0	6	0	2	0.007	3	0	3	NC	6	NC	6
118			min	0	3	-0.001	4	0.004	6	0	6	NC	1	6280.593	3
119		5	max	0	6	0	2	0.008	3	0	3	NC	6	NC	6
120			min	0	3	-0.002	4	0.004	6	0	6	NC	1	5759.958	3
121	M101	1	max	0	6	0	6	0	6	0	4	NC	6	NC	6
122			min	0	1	0	1	0	1	0	2	NC	1	NC	1
123		2	max	0	6	0.001	3	0.003	2	0	4	NC	6	NC	6
124			min	0	3	0	6	0.002	5	0	2	NC	1	NC	1
125		3	max	0	6	0.002	3	0.005	2	0	4	NC	6	NC	6
126			min	0	3	0.001	6	0.002	5	0	2	NC	1	8882.982	2
127		4	max	0	6	0.002	3	0.005	2	0	4	NC	6	NC	6
128			min	0	3	0.001	6	0.003	5	0	2	NC	1	8335.497	2
129		5	max	0	6	0.001	2	0.007	3	0	4	NC	6	NC	6
130			min	-0.001	3	0	4	0.004	6	0	2	NC	1	6291.735	3
131	M102	1	max	0.02	3	0.013	3	0.024	3	0	1	NC	6	NC	6
132			min	0.009	6	0.008	1	0.016	1	0	3	NC	1	NC	1
133		2	max	0.02	3	0.042	3	0.072	3	0	1	3099.153	1	1940.509	1
134			min	0.009	6	0.022	1	0.038	1	0	3	1508.001	3	922.283	3
135		3	max	0.02	3	0.071	3	0.134	3	0	1	1549.577	1	866.31	1
136			min	0.009	6	0.037	1	0.067	1	0	3	754	3	381.517	6
137		4	max	0.02	3	0.1	3	0.221	6	0	1	1033.051	1	540.679	1
138			min	0.009	6	0.051	1	0.097	1	0	3	502.667	3	215.996	6
139		5	max	0.02	3	0.129	3	0.317	6	0	1	774.788	1	390.553	1
140			min	0.009	6	0.065	1	0.128	1	0	3	377	3	146.577	6
141	M103	1	max	0.024	3	0.014	3	0.027	3	0	6	NC	6	NC	6
142			min	0.009	6	0.009	1	0.017	1	0	3	NC	1	NC	1
143		2	max	0.024	3	0.043	3	0.078	3	0	6	3037.319	1	1825.36	1
144			min	0.009	6	0.023	1	0.041	1	0	3	1477.72	3	872.045	3
145		3	max	0.024	3	0.073	3	0.142	3	0	6	1518.66	1	820.117	1
146			min	0.009	6	0.038	1	0.071	1	0	3	738.86	3	384.097	3
147		4	max	0.024	3	0.103	3	0.215	3	0	6	1012.44	1	513.597	1
148			min	0.009	6	0.052	1	0.103	1	0	3	492.573	3	234.719	3
149		5	max	0.024	3	0.133	3	0.293	3	0	6	759.33	1	371.678	1
150			min	0.009	6	0.067	1	0.136	1	0	3	369.43	3	165.248	3
151	M104	1	max	0.027	3	0.012	3	0.026	3	0	3	NC	6	NC	6
152			min	0.011	6	0.008	1	0.017	1	0	1	NC	1	NC	1
153		2	max	0.027	3	0.038	3	0.077	3	0	3	3517.219	1	1778.945	1
154			min	0.011	6	0.02	1	0.041	1	0	1	1710.64	3	861.676	3
155		3	max	0.027	3	0.063	3	0.142	3	0	3	1758.609	1	801.329	1
156			min	0.011	6	0.033	1	0.072	1	0	1	855.32	3	380.068	3
157		4	max	0.027	3	0.089	3	0.216	3	0	3	1172.406	1	502.529	1
158			min	0.011	6	0.045	1	0.104	1	0	1	570.213	3	232.46	3
159		5	max	0.027	3	0.115	3	0.295	3	0	3	879.305	1	363.944	1
160			min	0.011	6	0.058	1	0.138	1	0	1	427.66	3	163.755	3

Envelope Member Section Deflections - Strength (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
161	M108	1	max	0	5	0	2	0	6	0	6	NC	6	NC	6
162			min	0	2	0	6	0	1	0	2	NC	1	NC	1
163		2	max	0	5	0.001	2	0.006	2	0	6	NC	6	NC	6
164			min	0	2	0	4	0.001	5	0	2	NC	1	NC	1
165		3	max	0	5	0.001	5	0.024	2	0	6	NC	6	7164.473	6
166			min	0	2	0	4	0.013	6	0	2	NC	1	3802.06	2
167		4	max	0	5	0.001	5	0.096	3	0	6	NC	6	1840.829	6
168			min	0	2	0	4	0.05	6	0	2	NC	1	960.062	3
169		5	max	0	5	0.001	5	0.19	3	0	6	NC	6	933.082	6
170			min	0	2	0	1	0.099	6	0	2	NC	1	484.85	3
171	M111	1	max	0.01	3	0.007	3	0.014	3	0	1	NC	6	NC	6
172			min	0.005	1	0.005	1	0.009	1	-0.001	3	NC	1	NC	1
173		2	max	0.01	3	0.03	3	0.051	3	0	1	3975.116	1	2502.96	1
174			min	0.005	1	0.016	1	0.027	1	-0.001	3	1934.561	3	1179.215	3
175		3	max	0.01	3	0.053	3	0.102	3	0	1	1987.558	1	1083.754	1
176			min	0.005	1	0.027	1	0.05	1	-0.001	3	967.281	3	498.481	3
177		4	max	0.01	3	0.075	3	0.162	3	0	1	1325.039	1	665.725	1
178			min	0.005	1	0.038	1	0.075	1	-0.001	3	644.854	3	297.239	3
179		5	max	0.01	3	0.098	3	0.228	3	0	1	993.779	1	476.812	1
180			min	0.005	1	0.049	1	0.101	1	-0.001	3	483.64	3	205.903	3
181	M113	1	max	0.005	3	0.004	3	0.008	3	0	1	NC	6	NC	6
182			min	0.003	1	0.002	1	0.005	1	0	3	NC	1	NC	1
183		2	max	0.005	3	0.021	3	0.038	3	0	1	5346.412	1	3162.294	1
184			min	0.003	1	0.011	1	0.019	1	0	3	2602.016	3	1483.191	3
185		3	max	0.005	3	0.038	3	0.081	3	0	1	2673.206	1	1322.547	1
186			min	0.003	1	0.019	1	0.038	1	0	3	1301.008	3	602.956	3
187		4	max	0.005	3	0.054	3	0.133	3	0	1	1782.137	1	798.583	1
188			min	0.003	1	0.027	1	0.06	1	0	3	867.339	3	351.754	3
189		5	max	0.005	3	0.071	3	0.191	3	0	1	1336.603	1	566.874	1
190			min	0.003	1	0.035	1	0.083	1	0	3	650.504	3	240.3	3
191	M47	1	max	0.002	3	0.001	3	0.003	3	0	1	NC	6	NC	6
192			min	0.001	1	0.001	1	0.002	1	0	3	NC	1	NC	1
193		2	max	0.002	3	0.01	3	0.023	3	0	1	9690.531	1	4629.663	1
194			min	0.001	1	0.005	1	0.011	1	0	3	4716.219	3	2158.326	3
195		3	max	0.002	3	0.02	3	0.057	3	0	1	4845.266	1	1837.08	6
196			min	0.001	1	0.01	1	0.026	6	0	3	2358.109	3	808.607	3
197		4	max	0.002	3	0.029	3	0.1	3	0	1	3230.177	1	1089.689	6
198			min	0.001	1	0.014	1	0.042	6	0	3	1572.073	3	452.449	3
199		5	max	0.002	3	0.038	3	0.149	3	0	1	2422.633	1	766.557	6
200			min	0.001	1	0.019	1	0.059	6	0	3	1179.055	3	301.399	3
201	M49	1	max	0	6	0	6	0	6	0	6	NC	6	NC	6
202			min	0	1	0	1	0	1	0	3	NC	1	NC	1
203		2	max	0	2	0	5	0.003	2	0	6	NC	6	NC	6
204			min	0	5	0	2	0.002	6	0	3	NC	1	NC	1
205		3	max	0	2	0	5	0.005	2	0	6	NC	6	NC	6
206			min	0	5	0	2	0.003	6	0	3	NC	1	9065.905	2
207		4	max	0	2	0	6	0.004	2	0	6	NC	6	NC	6
208			min	0	5	-0.001	4	0.002	6	0	3	NC	1	NC	1
209		5	max	0	2	0	6	0.003	3	0	6	NC	6	NC	6
210			min	0	5	-0.001	4	0.002	6	0	3	NC	1	NC	1
211	M55	1	max	0	6	0	6	0	6	0	6	NC	6	NC	6
212			min	0	1	0	1	0	1	0	3	NC	1	NC	1
213		2	max	0	2	0	5	0.003	2	0	6	NC	6	NC	6
214			min	0	5	0	2	0.001	6	0	3	NC	1	NC	1
215		3	max	0	2	0	5	0.004	2	0	6	NC	6	NC	6

Envelope Member Section Deflections - Strength (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
216			min	0	5	0	2	0.002	6	0	3	NC	1	NC	1
217		4	max	0	2	0	5	0.002	2	0	6	NC	6	NC	6
218			min	0	5	0	2	0.001	6	0	3	NC	1	NC	1
219		5	max	0	2	0	2	0.001	3	0	6	NC	6	NC	6
220			min	0	5	0	4	0.001	6	0	3	NC	1	NC	1
221	M91	1	max	0	3	0	6	0	6	0	6	NC	6	NC	6
222			min	0	6	0	2	0	1	0	2	NC	1	NC	1
223		2	max	0	3	0	5	0.002	2	0	6	NC	6	NC	6
224			min	0	6	0	1	0.001	5	0	2	NC	1	NC	1
225		3	max	0	3	0.001	5	0.003	2	0	6	NC	6	NC	6
226			min	0	6	0	6	0.001	5	0	2	NC	1	NC	1
227		4	max	0.001	3	0.001	5	0.004	2	0	6	NC	6	NC	6
228			min	0	6	0	6	0.001	5	0	2	NC	1	9949.121	2
229		5	max	0.001	3	0	5	0.009	3	0	6	NC	6	8337.31	6
230			min	0	6	-0.001	4	0.005	6	0	2	NC	1	5088.51	3
231	M79	1	max	0	6	0	6	0	6	0	6	NC	6	NC	6
232			min	0	1	0	1	0	1	0	1	NC	1	NC	1
233		2	max	-0.001	6	-0.002	6	0	2	0	4	NC	6	NC	6
234			min	-0.001	2	-0.004	3	0	5	0	2	NC	1	NC	1
235		3	max	-0.001	6	-0.004	6	0	6	0	4	NC	6	NC	6
236			min	-0.003	2	-0.008	3	0	3	0	2	NC	1	NC	1
237		4	max	-0.002	6	-0.004	6	0	6	0	3	NC	6	NC	6
238			min	-0.004	2	-0.007	4	0	3	0	1	NC	1	NC	1
239		5	max	-0.003	6	-0.004	6	0	4	0.001	3	NC	6	NC	6
240			min	-0.005	2	-0.007	3	0	2	0	6	NC	1	NC	1
241	M42	1	max	-0.003	6	-0.012	6	-0.009	6	-0.002	1	NC	6	NC	6
242			min	-0.006	3	-0.02	3	-0.018	3	-0.003	3	NC	1	NC	1
243		2	max	-0.002	6	-0.019	1	-0.01	6	-0.002	1	NC	6	NC	6
244			min	-0.004	3	-0.03	3	-0.027	3	-0.005	3	9012.577	3	9704.168	3
245		3	max	-0.002	6	-0.018	1	-0.009	6	-0.002	1	NC	6	NC	6
246			min	-0.003	3	-0.027	3	-0.02	3	-0.004	3	7618.677	3	NC	1
247		4	max	-0.001	6	-0.008	1	-0.004	1	-0.001	1	NC	6	NC	6
248			min	-0.002	3	-0.012	3	-0.008	3	-0.003	3	NC	1	NC	1
249		5	max	0	6	0	6	0	6	0	6	NC	6	NC	6
250			min	0	1	0	1	0	1	0	1	NC	1	NC	1
251	M30	1	max	0.002	2	-0.013	6	0.007	3	-0.001	6	NC	6	NC	6
252			min	0.001	6	-0.026	2	0.003	6	-0.002	3	NC	1	NC	1
253		2	max	0.002	2	-0.011	6	0.006	3	-0.001	6	NC	6	NC	6
254			min	0.001	6	-0.021	2	0.003	6	-0.002	3	NC	1	NC	1
255		3	max	0.002	2	-0.009	6	0.006	3	-0.001	6	NC	6	NC	6
256			min	0.001	6	-0.017	2	0.003	6	-0.001	3	NC	1	NC	1
257		4	max	0.002	2	-0.007	6	0.006	3	-0.001	6	NC	6	NC	6
258			min	0.001	6	-0.013	2	0.003	6	-0.001	3	NC	1	NC	1
259		5	max	0.002	2	-0.005	6	0.005	3	0	6	NC	6	NC	6
260			min	0.001	4	-0.009	3	0.002	6	-0.001	3	NC	1	NC	1
261	M31	1	max	0	5	0	2	0	6	0	6	NC	6	NC	6
262			min	0	2	0	6	0	1	0	2	NC	1	NC	1
263		2	max	0	5	0.001	2	0.009	2	0	6	NC	6	NC	6
264			min	0	2	0	4	0.003	5	0	2	NC	1	9951.785	2
265		3	max	0	5	0.001	5	0.029	2	0	6	NC	6	6114.69	6
266			min	0	2	0	4	0.015	6	0	2	NC	1	3163.358	2
267		4	max	0	5	0.001	5	0.1	3	0	6	NC	6	1771.754	6
268			min	0	2	0	4	0.052	6	0	2	NC	1	920.238	3
269		5	max	0	5	0.001	5	0.193	3	0	6	NC	6	917.98	6
270			min	0	2	0	4	0.1	6	0	2	NC	1	476.003	3

Envelope Member Section Deflections - Strength (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
271	M32	1	max	0.02	3	-0.004	1	0.018	3	0.001	3	4211.768	1	651.914	6
272			min	0.012	6	-0.006	3	0.009	6	0.001	1	2685.946	3	332.466	3
273		2	max	0.02	3	-0.004	1	0.012	3	0.001	3	5804.286	1	883.689	6
274			min	0.012	6	-0.006	3	0.006	6	0.001	1	3696.382	3	450.651	3
275		3	max	0.02	3	-0.003	1	0.007	3	0.001	3	9090.294	1	1347.198	6
276			min	0.012	6	-0.005	3	0.003	6	0.001	1	5781.169	3	687.042	3
277		4	max	0.02	3	-0.003	6	0.001	3	0.001	3	NC	6	2737.557	6
278			min	0.012	6	-0.004	3	0	6	0.001	1	NC	1	1396.222	3
279		5	max	0.02	3	-0.002	6	-0.002	1	0.001	3	NC	6	NC	6
280			min	0.012	6	-0.004	3	-0.004	3	0	1	NC	1	NC	1
281	M33	1	max	0.002	2	-0.017	6	0.009	3	-0.001	6	8486.391	6	NC	6
282			min	0.001	6	-0.033	2	0.005	6	-0.002	3	3224.46	2	NC	1
283		2	max	0.002	2	-0.015	6	0.009	3	-0.001	6	NC	6	NC	6
284			min	0.001	6	-0.028	3	0.005	6	-0.002	3	5079.834	2	NC	1
285		3	max	0.002	2	-0.013	6	0.009	3	-0.001	6	NC	6	NC	6
286			min	0.001	6	-0.023	3	0.005	6	-0.002	3	NC	1	NC	1
287		4	max	0.002	2	-0.011	6	0.011	3	-0.001	6	NC	6	NC	6
288			min	0.001	6	-0.02	3	0.005	6	-0.003	3	NC	1	NC	1
289		5	max	0.003	2	-0.012	6	0.018	3	-0.001	6	NC	6	NC	1
290			min	0.001	6	-0.02	3	0.009	1	-0.003	3	NC	1	4792.871	3

Member Point Loads (BLC 6 : RAILING)

	Member Label	Direction	Magnitude [k, k-ft]	Location [(in, %)]
1	M60	My	-0.175	%100
2	M68	My	-0.175	%100
3	M78	My	-0.175	%100
4	M89	My	-0.175	%100
5	M102	My	-0.175	%100
6	M103	My	-0.175	%100
7	M104	My	-0.175	%100
8	M111	My	-0.175	%100
9	M113	My	-0.175	%100
10	M47	My	-0.175	%100
11	M75	My	-0.175	%100
12	M97	My	-0.175	%100
13	M98	My	-0.175	%100
14	M96	My	-0.175	%100
15	M101	My	-0.175	%100
16	M95	My	-0.175	%100
17	M70	My	-0.175	%100
18	M49	My	-0.175	%100
19	M55	My	-0.175	%100
20	M91	My	-0.175	%100

Member Distributed Loads (BLC 2 : SI)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	M113	Y	-0.04	-0.04	0	%100
2	M70	Y	-0.04	-0.04	0	%100
3	M95	Y	-0.04	-0.04	0	%100
4	M97	Y	-0.04	-0.04	0	%100
5	M75	Y	-0.04	-0.04	0	%100
6	M98	Y	-0.04	-0.04	0	%100
7	M96	Y	-0.04	-0.04	0	%100

Member Distributed Loads (BLC 2 : SI) (Continued)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
8	M101	Y	-0.04	-0.04	0	%100
9	M69	Y	-0.04	-0.04	0	%100
10	M85	Y	-0.04	-0.04	0	%100
11	M108	Y	-0.04	-0.04	0	%100
12	M89	Y	-0.04	-0.04	0	%100
13	M60	Y	-0.04	-0.04	0	%100
14	M104	Y	-0.04	-0.04	0	%100
15	M68	Y	-0.04	-0.04	0	%100
16	M103	Y	-0.04	-0.04	0	%100
17	M102	Y	-0.04	-0.04	0	%100
18	M78	Y	-0.04	-0.04	0	%100
19	M111	Y	-0.04	-0.04	0	%100
20	M47	Y	-0.04	-0.04	0	%100
21	M49	Y	-0.04	-0.04	0	%100
22	M55	Y	-0.04	-0.04	0	%100
23	M91	Y	-0.04	-0.04	0	%100
24	M31	Y	-0.04	-0.04	0	%100

Member Distributed Loads (BLC 3 : LL)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	M70	Y	-0.04	-0.04	0	%100
2	M95	Y	-0.04	-0.04	0	%100
3	M97	Y	-0.04	-0.04	0	%100
4	M75	Y	-0.04	-0.04	0	%100
5	M98	Y	-0.04	-0.04	0	%100
6	M96	Y	-0.04	-0.04	0	%100
7	M101	Y	-0.04	-0.04	0	%100
8	M69	Y	-0.04	-0.04	0	%100
9	M108	Y	-0.04	-0.04	0	%100
10	M85	Y	-0.04	-0.04	0	%100
11	M89	Y	-0.04	-0.04	0	%100
12	M60	Y	-0.04	-0.04	0	%100
13	M104	Y	-0.04	-0.04	0	%100
14	M68	Y	-0.04	-0.04	0	%100
15	M103	Y	-0.04	-0.04	0	%100
16	M102	Y	-0.04	-0.04	0	%100
17	M78	Y	-0.04	-0.04	0	%100
18	M111	Y	-0.04	-0.04	0	%100
19	M113	Y	-0.04	-0.04	0	%100
20	M47	Y	-0.04	-0.04	0	%100
21	M49	Y	-0.04	-0.04	0	%100
22	M55	Y	-0.04	-0.04	0	%100
23	M91	Y	-0.04	-0.04	0	%100
24	M31	Y	-0.04	-0.04	0	%100

Member Distributed Loads (BLC 4 : 0.5LL IN)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	M70	Y	-0.04	-0.04	%50	%100
2	M95	Y	-0.04	-0.04	%50	%100
3	M97	Y	-0.04	-0.04	%50	%100
4	M75	Y	-0.04	-0.04	%50	%100
5	M98	Y	-0.04	-0.04	%50	%100
6	M96	Y	-0.04	-0.04	%50	%100

Member Distributed Loads (BLC 4 : 0.5LL IN) (Continued)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
7	M101	Y	-0.04	-0.04	%50	%100
8	M69	Y	-0.04	-0.04	%25	%75
9	M85	Y	-0.04	-0.04	%25	%75
10	M108	Y	-0.04	-0.04	%25	%75
11	M89	Y	-0.04	-0.04	0	%50
12	M60	Y	-0.04	-0.04	0	%50
13	M104	Y	-0.04	-0.04	0	%50
14	M68	Y	-0.04	-0.04	0	%50
15	M103	Y	-0.04	-0.04	0	%50
16	M102	Y	-0.04	-0.04	0	%50
17	M78	Y	-0.04	-0.04	0	%50
18	M111	Y	-0.04	-0.04	0	%50
19	M113	Y	-0.04	-0.04	0	%50
20	M47	Y	-0.04	-0.04	0	%50
21	M49	Y	-0.04	-0.04	%50	%100
22	M55	Y	-0.04	-0.04	%50	%100
23	M91	Y	-0.04	-0.04	%50	%100
24	M31	Y	-0.04	-0.04	%25	%75

Member Distributed Loads (BLC 7 : 0.5LL OUT)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/in]	End Magnitude [k/ft, F, ksf, k-ft/in]	Start Location [(in, %)]	End Location [(in, %)]
1	M70	Y	-0.04	-0.04	0	%50
2	M95	Y	-0.04	-0.04	0	%50
3	M97	Y	-0.04	-0.04	0	%50
4	M75	Y	-0.04	-0.04	0	%50
5	M98	Y	-0.04	-0.04	0	%50
6	M96	Y	-0.04	-0.04	0	%50
7	M101	Y	-0.04	-0.04	0	%50
8	M69	Y	-0.04	-0.04	0	%25
9	M85	Y	-0.04	-0.04	0	%25
10	M108	Y	-0.04	-0.04	0	%25
11	M89	Y	-0.04	-0.04	%50	%100
12	M60	Y	-0.04	-0.04	%50	%100
13	M104	Y	-0.04	-0.04	%50	%100
14	M68	Y	-0.04	-0.04	%50	%100
15	M103	Y	-0.04	-0.04	%50	%100
16	M102	Y	-0.04	-0.04	%50	%100
17	M78	Y	-0.04	-0.04	%50	%100
18	M111	Y	-0.04	-0.04	%50	%100
19	M113	Y	-0.04	-0.04	%50	%100
20	M108	Y	-0.04	-0.04	%75	%100
21	M85	Y	-0.04	-0.04	%75	%100
22	M69	Y	-0.04	-0.04	%75	%100
23	M47	Y	-0.04	-0.04	%50	%100
24	M49	Y	-0.04	-0.04	0	%50
25	M55	Y	-0.04	-0.04	0	%50
26	M91	Y	-0.04	-0.04	0	%50
27	M31	Y	-0.04	-0.04	0	%25
28	M31	Y	-0.04	-0.04	%75	%100

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal	Point	Distributed
1	DL	DL	-1			
2	SI	DL				24
3	LL	LL				24
4	0.5LL IN	LL				24
5	PL	LL		1		
6	RAILING	OL1			20	
7	0.5LL OUT	LL				28

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL+SI	Yes	Y	1	1.4	2	1.4										
2	DL+SI+LL	Yes	Y	1	1.2	2	1.2	3	1.6								
3	DL+SI+LL+RAILING	Yes	Y	1	1.2	2	1.2	3	1.6					6	1.6		
4	DL+SI+0.5LL IN+RAILING	Yes	Y	1	1.2	2	1.2			4	1.6			6	1.6		
5	DL+SI+0.5LL OUT+RAILING	Yes	Y	1	1.2	2	1.2							6	1.6	7	1.6
6	DL+SI+PL	Yes	Y	1	1.2	2	1.2					5	1.6				

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	DL+SI		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	DL+SI+LL		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	DL+SI+LL+RAILING		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	DL+SI+0.5LL IN+RAILING		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	DL+SI+0.5LL OUT+RAILING		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	DL+SI+PL		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [$1e^{5}$°F^{-1}]	Density [k/ft^3]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A992	29000	11154	0.3	0.65	0.49	50	1.1	58	1.2
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	58	1.2
4	A500 Gr.B RND	29000	11154	0.3	0.65	0.527	42	1.3	58	1.1
5	A500 Gr.B Rect	29000	11154	0.3	0.65	0.527	46	1.2	58	1.1
6	A53 Gr.B	29000	11154	0.3	0.65	0.49	35	1.5	58	1.2
7	A1085	29000	11154	0.3	0.65	0.49	50	1.1	58	1.2

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in^2]	Iyy [in^4]	Izz [in^4]	J [in^4]
1	stringer	HSS10X4X6	Beam	Tube	A500 Gr.B Rect	Typical	8.97	24.3	104	66.5
2	Tread	HSS10X2X4	Beam	Tube	A500 Gr.B Rect	Typical	5.24	3.67	52.5	12.2
3	plate	PLATE	Beam	None	A36 Gr.36	Typical	5	41.667	0.104	0.005
4	Beam	HSS6X4X4	Beam	Tube	A500 Gr.B Rect	Typical	4.3	11.1	20.9	23.6
5	Column	HSS10X4X6	Column	Tube	A500 Gr.B Rect	Typical	8.97	24.3	104	66.5

Beam Deflections

	LC	Member Label	Span	Location [in]	y' [in]	(n) L'/y' Ratio
1	1	M60	1	44	0	NC
2	1	M68	1	44	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [in]	y' [in]	(n) L'/y' Ratio
3	1	M69	1	38.333	0.001	NC
4	1	M70	1	44	0	NC
5	1	M75	1	44	0	NC
6	1	M78	1	44	0	NC
7	1	M85	1	60.375	0.001	NC
8	1	M89	1	44	0	NC
9	1	M95	1	44	0	NC
10	1	M96	1	44	-0.001	NC
11	1	M97	1	44	0	NC
12	1	M98	1	44	0	NC
13	1	M101	1	36.667	-0.001	NC
14	1	M102	1	44	0	NC
15	1	M103	1	44	0	NC
16	1	M104	1	44	0	NC
17	1	M108	1	92	0	NC
18	1	M111	1	44	0	NC
19	1	M113	1	44	0	NC
20	1	M47	1	44	0	NC
21	1	M49	1	44	0	NC
22	1	M55	1	44	0	NC
23	1	M91	1	39.417	-0.001	NC
24	1	M79	1	16.484	-0.001	NC
25	1	M42	1	131.868	0	NC
26	1	M30	1	44	0	NC
27	1	M31	1	92	0	NC
28	1	M33	1	0	-0.014	6267
29	2	M60	1	44	0	NC
30	2	M68	1	44	0	NC
31	2	M69	1	31.625	0.001	NC
32	2	M70	1	44	0.001	NC
33	2	M75	1	44	0	NC
34	2	M78	1	44	0	NC
35	2	M85	1	48.875	0.001	NC
36	2	M89	1	44	0	NC
37	2	M95	1	44	0.001	NC
38	2	M96	1	38.958	-0.001	NC
39	2	M97	1	44	0	NC
40	2	M98	1	44	0	NC
41	2	M101	1	31.625	-0.001	NC
42	2	M102	1	44	0	NC
43	2	M103	1	44	0	NC
44	2	M104	1	44	0	NC
45	2	M108	1	92	0	NC
46	2	M111	1	44	0	NC
47	2	M113	1	44	0	NC
48	2	M47	1	44	0	NC
49	2	M49	1	44	0	NC
50	2	M55	1	44	0	NC
51	2	M91	1	32.542	-0.001	NC
52	2	M79	1	13.736	-0.001	NC
53	2	M42	1	54.945	-0.018	7449
54	2	M30	1	44	0	NC
55	2	M31	1	92	-0.001	NC
56	2	M33	1	0	-0.022	4048
57	3	M60	1	44	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [in]	y' [in]	(n) L'/y' Ratio
58	3	M68	1	44	0	NC
59	3	M69	1	29.708	0.001	NC
60	3	M70	1	44	0	NC
61	3	M75	1	44	-0.001	NC
62	3	M78	1	44	0	NC
63	3	M85	1	44.083	0.001	NC
64	3	M89	1	44	0	NC
65	3	M95	1	44	0	NC
66	3	M96	1	33	-0.001	NC
67	3	M97	1	44	0	NC
68	3	M98	1	39.875	-0.001	NC
69	3	M101	1	28.875	-0.001	NC
70	3	M102	1	44	0	NC
71	3	M103	1	44	0	NC
72	3	M104	1	44	0	NC
73	3	M108	1	92	0	NC
74	3	M111	1	44	0	NC
75	3	M113	1	44	0	NC
76	3	M47	1	44	0	NC
77	3	M49	1	44	0	NC
78	3	M55	1	44	0	NC
79	3	M91	1	31.167	-0.001	NC
80	3	M79	1	10.989	-0.001	NC
81	3	M42	1	54.945	-0.018	7270
82	3	M30	1	44	0	NC
83	3	M31	1	92	0	NC
84	3	M33	1	0	-0.022	4037
85	4	M60	1	44	0	NC
86	4	M68	1	44	0	NC
87	4	M69	1	32.583	0.001	NC
88	4	M70	1	44	0	NC
89	4	M75	1	44	-0.001	NC
90	4	M78	1	44	0	NC
91	4	M85	1	45.042	0.001	NC
92	4	M89	1	44	0	NC
93	4	M95	1	44	0	NC
94	4	M96	1	35.75	-0.001	NC
95	4	M97	1	44	0	NC
96	4	M98	1	42.625	-0.001	NC
97	4	M101	1	31.625	-0.001	NC
98	4	M102	1	44	0	NC
99	4	M103	1	44	0	NC
100	4	M104	1	44	0	NC
101	4	M108	1	92	0.001	NC
102	4	M111	1	44	0	NC
103	4	M113	1	44	0	NC
104	4	M47	1	44	0	NC
105	4	M49	1	44	0	NC
106	4	M55	1	44	0	NC
107	4	M91	1	35.292	-0.001	NC
108	4	M79	1	12.363	-0.001	NC
109	4	M42	1	54.945	-0.014	9129
110	4	M30	1	44	0	NC
111	4	M31	1	92	0.001	NC
112	4	M33	1	0	-0.017	5171

Beam Deflections (Continued)

	LC	Member Label	Span	Location [in]	y' [in]	(n) L'/y' Ratio
113	5	M60	1	44	0	NC
114	5	M68	1	44	0	NC
115	5	M69	1	32.583	0.001	NC
116	5	M70	1	44	0	NC
117	5	M75	1	44	-0.001	NC
118	5	M78	1	44	0	NC
119	5	M85	1	46.958	0.001	NC
120	5	M89	1	44	0	NC
121	5	M95	1	44	0	NC
122	5	M96	1	33.458	-0.001	NC
123	5	M97	1	44	-0.001	NC
124	5	M98	1	39.417	-0.001	NC
125	5	M101	1	29.333	-0.001	NC
126	5	M102	1	44	0	NC
127	5	M103	1	44	0	NC
128	5	M104	1	44	0	NC
129	5	M108	1	92	0	NC
130	5	M111	1	44	0	NC
131	5	M113	1	44	0	NC
132	5	M47	1	44	0	NC
133	5	M49	1	44	0	NC
134	5	M55	1	44	0	NC
135	5	M91	1	32.542	-0.001	NC
136	5	M79	1	13.736	-0.001	NC
137	5	M42	1	54.945	-0.014	9229
138	5	M30	1	44	0	NC
139	5	M31	1	92	0	NC
140	5	M33	1	0	-0.017	5215
141	6	M60	1	44	0	NC
142	6	M68	1	44	0	NC
143	6	M69	1	36.417	0.001	NC
144	6	M70	1	44	0	NC
145	6	M75	1	44	0	NC
146	6	M78	1	44	0	NC
147	6	M85	1	55.583	0.001	NC
148	6	M89	1	44	0	NC
149	6	M95	1	44	0	NC
150	6	M96	1	44	-0.001	NC
151	6	M97	1	44	0	NC
152	6	M98	1	44	0	NC
153	6	M101	1	37.125	-0.001	NC
154	6	M102	1	44	0	NC
155	6	M103	1	44	0	NC
156	6	M104	1	44	0	NC
157	6	M108	1	92	0	NC
158	6	M111	1	44	0	NC
159	6	M113	1	44	0	NC
160	6	M47	1	44	0	NC
161	6	M49	1	44	0	NC
162	6	M55	1	44	0	NC
163	6	M91	1	40.792	-0.001	NC
164	6	M79	1	17.857	-0.001	NC
165	6	M42	1	56.319	-0.013	9841
166	6	M30	1	44	0	NC
167	6	M31	1	92	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [in]	y' [in]	(n) L'/y' Ratio
168	6	M33	1	0	-0.013	6845

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

	LC	Member	Shape	UC Max	Loc[in]	Shear UC	Loc[in]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
1	1	M60	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
2	1	M68	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
3	1	M69	HSS10X2X4	0.147	44.083	0.042	47.917	y	96.224	216.936	11.824	49.68	2.098	H1-1b
4	1	M70	HSS10X2X4	0.01	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
5	1	M75	HSS10X2X4	0.009	18.792	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
6	1	M78	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
7	1	M85	HSS10X2X4	0.076	44.083	0.02	48.875	z	96.224	216.936	11.824	49.68	2.275	H1-1b
8	1	M89	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
9	1	M95	HSS10X2X4	0.01	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
10	1	M96	HSS10X2X4	0.017	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
11	1	M97	HSS10X2X4	0.009	19.708	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
12	1	M98	HSS10X2X4	0.01	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
13	1	M101	HSS10X2X4	0.029	44	0.013	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
14	1	M102	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
15	1	M103	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
16	1	M104	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
17	1	M108	HSS10X2X4	0.057	47.917	0.02	48.875	z	96.224	216.936	11.824	49.68	2.325	H1-1b
18	1	M111	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
19	1	M113	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
20	1	M47	HSS10X2X4	0.047	0	0.02	0	z	176.967	216.936	11.824	49.55	1	H1-1b
21	1	M49	HSS10X2X4	0.011	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
22	1	M55	HSS10X2X4	0.011	44	0.012	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
23	1	M91	HSS10X2X4	0.033	44	0.016	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
24	1	M79	HSS10X4X6	0.064	131.868	0.043	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
25	1	M42	HSS10X4X6	0.063	131.868	0.068	131.868	y	241.139	371.358	48.3	93.15	2.312	H1-1b
26	1	M30	HSS10X4X6	0.037	33	0.047	44	z	353.928	371.358	48.3	93.15	1.339	H1-1b
27	1	M31	HSS10X2X4	0.051	44.083	0.02	48.875	z	96.224	216.936	11.824	49.68	2.259	H1-1b
28	1	M32	HSS10X4X6	0.057	7.249	0.055	7.249	y	370.874	371.358	48.3	93.15	1.481	H1-1b
29	1	M33	HSS10X4X6	0.088	44	0.025	33	y	353.928	371.358	48.3	93.15	2.016	H1-1b
30	2	M60	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
31	2	M68	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
32	2	M69	HSS10X2X4	0.237	44.083	0.064	47.917	y	96.224	216.936	11.824	49.68	2.086	H1-1b
33	2	M70	HSS10X2X4	0.017	44	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
34	2	M75	HSS10X2X4	0.014	18.792	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
35	2	M78	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
36	2	M85	HSS10X2X4	0.123	44.083	0.032	48.875	z	96.224	216.936	11.824	49.68	2.283	H1-1b
37	2	M89	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
38	2	M95	HSS10X2X4	0.016	44	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
39	2	M96	HSS10X2X4	0.029	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
40	2	M97	HSS10X2X4	0.015	44	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
41	2	M98	HSS10X2X4	0.017	44	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
42	2	M101	HSS10X2X4	0.047	44	0.021	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
43	2	M102	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
44	2	M103	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
45	2	M104	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
46	2	M108	HSS10X2X4	0.093	47.917	0.032	48.875	z	96.224	216.936	11.824	49.68	2.287	H1-1b
47	2	M111	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
48	2	M113	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
49	2	M47	HSS10X2X4	0.077	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
50	2	M49	HSS10X2X4	0.018	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
51	2	M55	HSS10X2X4	0.018	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b

AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[in]	Shear UC	Loc[in]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
52	2	M91	HSS10X2X4	0.053	44	0.026	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
53	2	M79	HSS10X4X6	0.1	131.868	0.07	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
54	2	M42	HSS10X4X6	0.097	131.868	0.109	131.868	y	241.139	371.358	48.3	93.15	2.322	H1-1b
55	2	M30	HSS10X4X6	0.058	33	0.076	44	z	353.928	371.358	48.3	93.15	1.334	H1-1b
56	2	M31	HSS10X2X4	0.083	44.083	0.032	48.875	z	96.224	216.936	11.824	49.68	2.25	H1-1b
57	2	M32	HSS10X4X6	0.09	7.249	0.086	7.249	y	370.874	371.358	48.3	93.15	1.472	H1-1b
58	2	M33	HSS10X4X6	0.139	44	0.04	44	y	353.928	371.358	48.3	93.15	2.011	H1-1b
59	3	M60	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
60	3	M68	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
61	3	M69	HSS10X2X4	0.269	44.083	0.06	47.917	y	96.224	216.936	11.824	49.68	2.02	H1-1b
62	3	M70	HSS10X2X4	0.019	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
63	3	M75	HSS10X2X4	0.022	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
64	3	M78	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
65	3	M85	HSS10X2X4	0.139	44.083	0.032	48.875	z	96.224	216.936	11.824	49.68	2.196	H1-1b
66	3	M89	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
67	3	M95	HSS10X2X4	0.018	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
68	3	M96	HSS10X2X4	0.041	44	0.021	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
69	3	M97	HSS10X2X4	0.019	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
70	3	M98	HSS10X2X4	0.029	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
71	3	M101	HSS10X2X4	0.06	44	0.023	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
72	3	M102	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
73	3	M103	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
74	3	M104	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
75	3	M108	HSS10X2X4	0.104	44.083	0.032	48.875	z	96.224	216.936	11.824	49.68	2.437	H1-1b
76	3	M111	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
77	3	M113	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
78	3	M47	HSS10X2X4	0.1	0	0.033	0	z	176.967	216.936	11.824	49.55	1	H1-1b
79	3	M49	HSS10X2X4	0.02	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
80	3	M55	HSS10X2X4	0.02	44	0.02	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
81	3	M91	HSS10X2X4	0.067	44	0.029	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
82	3	M79	HSS10X4X6	0.114	131.868	0.083	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
83	3	M42	HSS10X4X6	0.102	131.868	0.136	131.868	y	241.139	371.358	48.3	93.15	2.306	H1-1b
84	3	M30	HSS10X4X6	0.064	33	0.079	44	z	353.928	371.358	48.3	93.15	1.331	H1-1b
85	3	M31	HSS10X2X4	0.092	44.083	0.032	48.875	z	96.224	216.936	11.824	49.68	2.553	H1-1b
86	3	M32	HSS10X4X6	0.104	7.249	0.1	7.249	y	370.874	371.358	48.3	93.15	1.504	H1-1b
87	3	M33	HSS10X4X6	0.153	44	0.053	44	z	353.928	371.358	48.3	93.15	2.029	H1-1b
88	4	M60	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
89	4	M68	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
90	4	M69	HSS10X2X4	0.188	44.083	0.047	47.917	y	96.224	216.936	11.824	49.68	2.029	H1-1b
91	4	M70	HSS10X2X4	0.016	44	0.018	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
92	4	M75	HSS10X2X4	0.019	44	0.017	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
93	4	M78	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
94	4	M85	HSS10X2X4	0.095	44.083	0.024	48.875	z	96.224	216.936	11.824	49.68	2.165	H1-1b
95	4	M89	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
96	4	M95	HSS10X2X4	0.016	44	0.017	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
97	4	M96	HSS10X2X4	0.032	44	0.018	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
98	4	M97	HSS10X2X4	0.017	44	0.017	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
99	4	M98	HSS10X2X4	0.023	44	0.018	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
100	4	M101	HSS10X2X4	0.045	44	0.019	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
101	4	M102	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
102	4	M103	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
103	4	M104	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
104	4	M108	HSS10X2X4	0.07	44.083	0.024	48.875	z	96.224	216.936	11.824	49.68	2.468	H1-1b
105	4	M111	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
106	4	M113	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b

AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[in]	Shear UC	Loc[in]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
107	4	M47	HSS10X2X4	0.073	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
108	4	M49	HSS10X2X4	0.017	44	0.018	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
109	4	M55	HSS10X2X4	0.017	44	0.018	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
110	4	M91	HSS10X2X4	0.049	44	0.024	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
111	4	M79	HSS10X4X6	0.087	131.868	0.057	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
112	4	M42	HSS10X4X6	0.079	131.868	0.099	131.868	y	241.139	371.358	48.3	93.15	2.293	H1-1b
113	4	M30	HSS10X4X6	0.05	33	0.053	44	z	353.928	371.358	48.3	93.15	1.349	H1-1b
114	4	M31	HSS10X2X4	0.062	44.083	0.024	48.875	z	96.224	216.936	11.824	49.68	2.69	H1-1b
115	4	M32	HSS10X4X6	0.08	7.249	0.076	7.249	y	370.874	371.358	48.3	93.15	1.52	H1-1b
116	4	M33	HSS10X4X6	0.116	44	0.041	44	y	353.928	371.358	48.3	93.15	2.05	H1-1b
117	5	M60	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
118	5	M68	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
119	5	M69	HSS10X2X4	0.238	44.083	0.045	47.917	y	96.224	216.936	11.824	49.68	1.989	H1-1b
120	5	M70	HSS10X2X4	0.016	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
121	5	M75	HSS10X2X4	0.02	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
122	5	M78	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
123	5	M85	HSS10X2X4	0.125	44.083	0.025	48.875	z	96.224	216.936	11.824	49.68	2.184	H1-1b
124	5	M89	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
125	5	M95	HSS10X2X4	0.016	44	0.013	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
126	5	M96	HSS10X2X4	0.036	44	0.015	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
127	5	M97	HSS10X2X4	0.017	44	0.013	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
128	5	M98	HSS10X2X4	0.026	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
129	5	M101	HSS10X2X4	0.053	44	0.016	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
130	5	M102	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
131	5	M103	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
132	5	M104	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
133	5	M108	HSS10X2X4	0.094	44.083	0.025	48.875	z	96.224	216.936	11.824	49.68	2.405	H1-1b
134	5	M111	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
135	5	M113	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
136	5	M47	HSS10X2X4	0.091	0	0.025	0	z	176.967	216.936	11.824	49.55	1	H1-1b
137	5	M49	HSS10X2X4	0.017	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
138	5	M55	HSS10X2X4	0.016	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
139	5	M91	HSS10X2X4	0.061	44	0.022	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
140	5	M79	HSS10X4X6	0.096	131.868	0.075	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
141	5	M42	HSS10X4X6	0.083	131.868	0.122	131.868	z	241.139	371.358	48.3	93.15	2.302	H1-1b
142	5	M30	HSS10X4X6	0.053	33	0.07	44	z	353.928	371.358	48.3	93.15	1.313	H1-1b
143	5	M31	HSS10X2X4	0.083	44.083	0.025	48.875	z	96.224	216.936	11.824	49.68	2.499	H1-1b
144	5	M32	HSS10X4X6	0.087	7.249	0.085	7.249	y	370.874	371.358	48.3	93.15	1.515	H1-1b
145	5	M33	HSS10X4X6	0.126	44	0.049	44	z	353.928	371.358	48.3	93.15	2.023	H1-1b
146	6	M60	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
147	6	M68	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
148	6	M69	HSS10X2X4	0.149	44.083	0.047	44.083	z	96.224	216.936	11.824	49.68	2.027	H1-1b
149	6	M70	HSS10X2X4	0.009	44	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
150	6	M75	HSS10X2X4	0.008	18.792	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
151	6	M78	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
152	6	M85	HSS10X2X4	0.073	44.083	0.017	48.875	z	96.224	216.936	11.824	49.68	2.186	H1-1b
153	6	M89	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
154	6	M95	HSS10X2X4	0.008	44	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
155	6	M96	HSS10X2X4	0.016	44	0.011	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
156	6	M97	HSS10X2X4	0.008	19.708	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
157	6	M98	HSS10X2X4	0.009	44	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
158	6	M101	HSS10X2X4	0.027	44	0.011	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
159	6	M102	HSS10X2X4	0.189	0	0.049	0	z	176.967	216.936	11.824	49.55	1	H1-1b
160	6	M103	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
161	6	M104	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b

AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[in]	Shear UC	Loc[in]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
162	6	M108	HSS10X2X4	0.054	47.917	0.017	48.875	z	96.224	216.936	11.824	49.68	2.33	H1-1b
163	6	M111	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
164	6	M113	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
165	6	M47	HSS10X2X4	0.04	0	0.017	0	z	176.967	216.936	11.824	49.55	1	H1-1b
166	6	M49	HSS10X2X4	0.009	44	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
167	6	M55	HSS10X2X4	0.009	44	0.01	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
168	6	M91	HSS10X2X4	0.031	44	0.014	44	z	176.967	216.936	11.824	49.68	1.667	H1-1b
169	6	M79	HSS10X4X6	0.058	131.868	0.042	131.868	z	241.139	371.358	48.3	93.15	3	H1-1b
170	6	M42	HSS10X4X6	0.068	131.868	0.077	131.868	y	241.139	371.358	48.3	93.15	2.191	H1-1b
171	6	M30	HSS10X4X6	0.034	33	0.046	44	z	353.928	371.358	48.3	93.15	1.289	H1-1b
172	6	M31	HSS10X2X4	0.047	44.083	0.017	48.875	z	96.224	216.936	11.824	49.68	2.258	H1-1b
173	6	M32	HSS10X4X6	0.057	7.249	0.057	7.249	y	370.874	371.358	48.3	93.15	1.47	H1-1b
174	6	M33	HSS10X4X6	0.088	44	0.027	44	z	353.928	371.358	48.3	93.15	1.965	H1-1b

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M60	N88	N83	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
2	M68	N87	N1	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
3	M69	N79	N70	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
4	M70	N73	N45	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
5	M75	N75	N66	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
6	M78	N85	N80	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
7	M85	N103	N102	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
8	M89	N18	N84	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
9	M95	N52	N65	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
10	M96	N77	N69	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
11	M97	N74	N67	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
12	M98	N76	N68	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
13	M101	N78	N63	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
14	M102	N11	N81	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
15	M103	N86	N72	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
16	M104	N14	N82	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
17	M108	N107	N104	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
18	M111	N115	N118	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
19	M113	N119	N122	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
20	M47	N99	N111	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
21	M49	N134	N133	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
22	M55	N138	N137	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
23	M91	N143	N144	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
24	M79	N141	N144		stringer	Beam	Tube	A500 Gr.B Rect	Typical
25	M42	N18	N114		stringer	Beam	Tube	A500 Gr.B Rect	Typical
26	M30	N71	N144		stringer	Beam	Tube	A500 Gr.B Rect	Typical
27	M31	N64	N92	90	Tread	Beam	Tube	A500 Gr.B Rect	Typical
28	M32	N18	N93		Column	Column	Tube	A500 Gr.B Rect	Typical
29	M33	N30	N93		stringer	Beam	Tube	A500 Gr.B Rect	Typical

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	N138	max	0.02	2	0.195	2	0.07	2	0	6	0	6	0	6
2		min	-0.024	5	0.102	6	-0.048	5	0	1	0	1	0	1
3	N134	max	0.041	2	0.201	2	0.044	2	0	6	0	6	0	6
4		min	-0.031	5	0.106	6	-0.023	5	0	1	0	1	0	1
5	N73	max	0.054	2	0.205	2	0.014	2	0	6	0	6	0	6

Envelope Node Reactions (Continued)

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
6		min	-0.034	5	0.108	6	-0.014	5	0	1	0	1	0	1
7	N52	max	0.055	2	0.208	2	-0.008	6	0	6	0	6	0	6
8		min	-0.039	5	0.11	6	-0.025	3	0	1	0	1	0	1
9	N74	max	0.044	2	0.209	2	-0.029	6	0	6	0	6	0	6
10		min	-0.052	5	0.111	6	-0.057	3	0	1	0	1	0	1
11	N75	max	0.016	2	0.207	2	-0.054	6	0	6	0	6	0	6
12		min	-0.078	5	0.11	6	-0.101	3	0	1	0	1	0	1
13	N76	max	-0.023	1	0.202	2	-0.106	6	0	6	0	6	0	6
14		min	-0.129	5	0.107	6	-0.197	3	0	1	0	1	0	1
15	N77	max	-0.082	6	0.192	2	-0.301	6	0	6	0	6	0	6
16		min	-0.23	3	0.101	6	-0.564	3	0	1	0	1	0	1
17	N78	max	-0.16	6	0.172	2	-0.859	6	0	6	0	6	0	6
18		min	-0.365	3	0.088	4	-1.575	3	0	1	0	1	0	1
19	N143	max	-0.103	6	0.112	2	1.057	5	0	6	0	6	0	6
20		min	-0.225	3	0.028	4	0.227	6	0	1	0	1	0	1
21	N79	max	0.239	3	0.066	2	-0.352	1	0	6	0	6	0	6
22		min	0.121	1	0.002	4	-0.831	3	0	1	0	1	0	1
23	N103	max	0.056	4	0.094	2	0.628	3	0	6	0	6	0	6
24		min	0.021	1	0.031	4	0.394	1	0	1	0	1	0	1
25	N107	max	0.005	4	0.093	2	0.581	3	0	6	0	6	0	6
26		min	-0.025	2	0.032	4	0.341	1	0	1	0	1	0	1
27	N64	max	0.002	4	0.097	2	0.479	3	0	6	0	6	0	6
28		min	-0.022	2	0.036	4	0.248	6	0	1	0	1	0	1
29	N141	max	-3.792	6	5.591	3	0.099	2	0.177	4	0.104	2	-1.278	6
30		min	-6.473	2	3.288	6	-0.077	5	-0.129	2	-0.129	4	-2.555	3
31	N114	max	6.982	3	7.667	3	0.763	3	4.518	3	4.174	3	-4.131	1
32		min	3.888	6	4.613	6	0.368	1	2.138	1	2.023	1	-6.359	3
33	Totals:	max	0	1	15.22	2	0	2						
34		min	0	3	9.044	6	0	6						

NODE 141

Weld Design For Tube

MATERIALS:

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

Weld and Section Data:

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

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

$d := 10.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 3.0$ Moment (Kips-ft)

$V_{weld} := 6.5$ Shear (Kips)

$P_{weld} := 5.6$ Tension or Compression (Kips)

$T_{weld} := 0.5$ Torsional Moment (kips-ft)

Check Weld Section :

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

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

$t_e = 0.04$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 2.93$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $A_{weld} = 1.12$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $J_{weld} = 18.30$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$ $f_b = 12.27$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$ $f_v = 10.47$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$ $f_a = 5.00$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{\left(f_b + f_a\right)^2 + f_v^2}$$ $f_{weld} = 20.19$ ksi

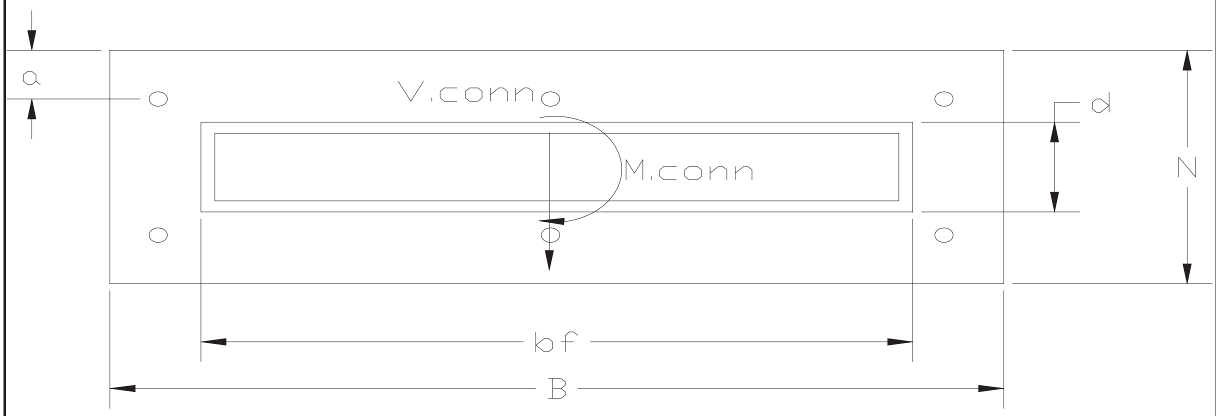
$$STRESS := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } F_v \ge f_{weld} \end{cases}$$ STRESS = "OK"

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

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

NODE 141

Moment-Resisting Bearing Type Plate Connection Design



Loads Data:

$M := 3.0$ Moment in Connection (kips-ft)

$P := 3.5$ Axial in Connection **Positive Down (Gravity)** (kips)

$V := 6.5$ Shear in Connection (kips)

Plate Data:

$B := 10.0$ Plate Width (in)

$N := 18.0$ Plate Lenght along Moment Direction (in)

$a := 1.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 10.0$ Depth of Column along Length (N) direction (in)

$b_f := 4.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate Material (Ksi)

$Fb_{plate} := 21.60$ Allowable Tensile Bending Stress of Plate Material (Ksi)
Fb=0.6*Fy .... for Steel
Fb=Fy/1.85......................for Stainless Steel

Allowable Support Stress Data:

$Fp_{supp} := 1.050$ **Allowable** compressive strength of support (ksi)
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

Concrete Bearing Stress:

$$e := \frac{12M}{\max(0.1, P)}$$

$e = 10.29$ in

$$f_b := \begin{cases} \text{"INCREASE PLATE"} & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} & \text{if } e < \frac{N}{6} \\ \frac{2P}{3 \cdot \left(\frac{N}{2} - e\right) \cdot B} & \text{if } \frac{N}{6} < e < \frac{N}{2} \\ Fp_{supp} & \text{if } e \geq \frac{N}{2} \wedge \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq Fp_{supp} \\ 0.00 & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq 0.00 \end{cases}$$

$f_b = 1.05$ ksi

Compression Length:

$$Kd := \text{if}\left[e \leq \frac{N}{6}, N, \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\frac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\frac{x^2}{3}\right) \leq M \cdot 12 \\ \quad x \leftarrow x + 0.00001 \end{array}\right]$$

$Kd = 0.73$

Tension in Bolts:

$$T_{bolts} := \begin{cases} 0.5 \cdot f_b \cdot Kd \cdot B - P \\ 0 & \text{if } e \leq \frac{N}{6} \end{cases}$$

$T_{bolts} = 0.32$ Kips

Tension per Bolt:

$$T_{Perbolts} := \frac{T_{bolts}}{6}$$

$T_{Perbolts} = 0.05$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2} \qquad m = 4.25 \quad in$$

$$n := \frac{B - 0.8 \cdot b_f}{2} \qquad n = 3.40 \quad in$$

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases} \qquad f_2 = 0.00 \quad ksi$$

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a) T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{(f_b - f_2) B \cdot m^2}{3}, (m - a) T_{bolts}\right] & \text{if } Kd > m \end{cases} \qquad M_{plate} = 15.31 \quad kips - in$$

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}} \qquad t_{plate} = 0.65 \quad in$$

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Rev1 Bottom Connection	Date:	4/3/2026
Fastening point:			

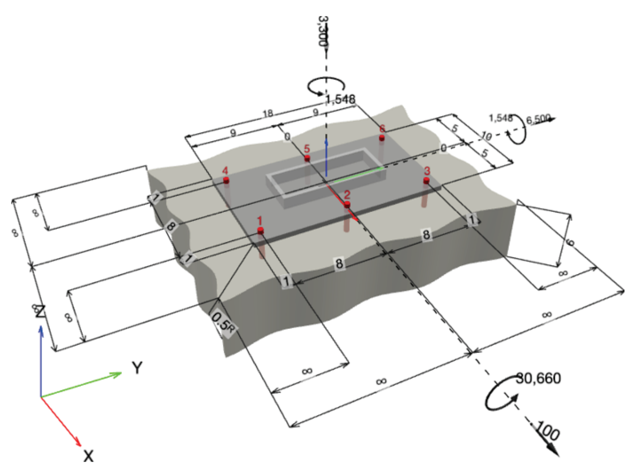
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 1/2 (3)**
Item number:	418072 KH-EZ 1/2"x3 1/2"
Specification text:	HILTI ∅ 1/2 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 3 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 2.160 in., h_{nom} = 3.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 10.000 in. x 18.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X4X.375; (L x W x T) = 10.000 in. x 4.000 in. x 0.375 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 6.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

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

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

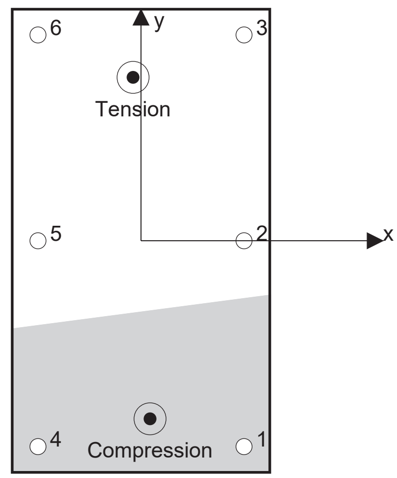


Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Rev1 Bottom Connection	Date:	4/3/2026
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -3,300; V_x = 100; V_y = 6,500; M_x = 30,660; M_y = 1,548; M_z = 1,548;	no	57

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,102	52	1,101
2	49	1,101	17	1,101
3	223	1,101	-19	1,101
4	0	1,067	52	1,066
5	72	1,066	17	1,066
6	245	1,066	-19	1,066

Max. concrete compressive strain:	0.03 [‰]
Max. concrete compressive stress:	138 [psi]
Resulting tension force in (x/y)=(-0.308/6.352):	590 [lb]
Resulting compression force in (x/y)=(0.351/-6.919):	3,890 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	245	11,778	3	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	590	4,066	15	OK

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

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3027

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

Variables

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

Calculations

N_{sa} [lb]
18,120

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,120	0.650	11,778	245

3.2 Concrete Breakout Failure

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

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

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

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

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

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

$$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$$ 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.]	$\psi_{c,N}$
2.160	0.308	2.352	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
3.750	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
167.96	41.99	0.913	0.579	1.000	1.000	2,956

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
6,256	0.650	4,066	590

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,102	5,547	20	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	6,501	11,552	57	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027

ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	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	1,102

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$	ACI 318-19 Eq. (17.7.3.1b)
$\phi\ V_{cpg} \geq V_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}} \right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.3.1)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$	ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
1	2.160	0.238	0.004	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	3.750	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
251.94	41.99	0.932	0.999	1.000	1.000	2,956

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
16,503	0.700	11,552	6,501

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.145	0.563	5/3	43	OK

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

6 Warnings

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

Fastening meets the design criteria!

NODE 114

Weld Design For Tube

MATERIALS:

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

Weld and Section Data:

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

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

$d := 10.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 6.0$ Moment (Kips-ft)

$V_{weld} := 8.0$ Shear (Kips)

$P_{weld} := 7.0$ Tension or Compression (Kips)

$T_{weld} := 4.6$ Torsional Moment (kips-ft)

Check Weld Section :

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

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

$t_e = 0.08$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 5.87$ in^3

$A_{weld} := 2\cdot(d + b)\cdot t_e$ $A_{weld} = 2.24$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $J_{weld} = 36.59$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$ $f_b = 12.27$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$ $f_v = 13.48$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$ $f_a = 3.12$ Ksi

Check Combined Stress :

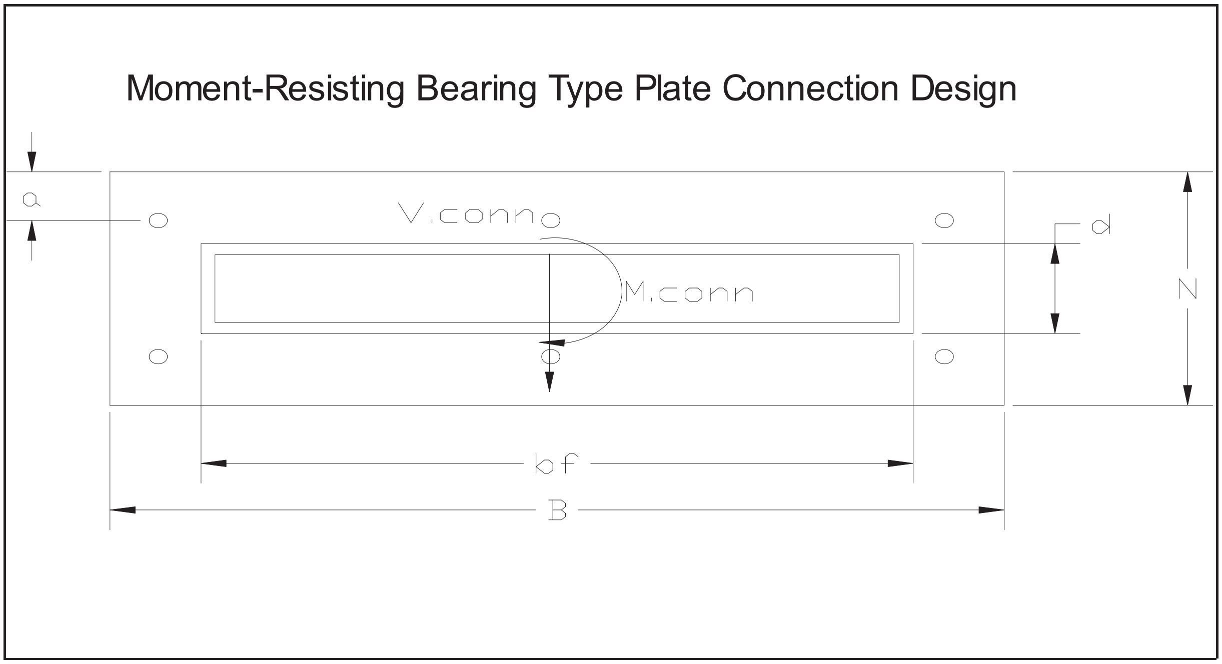
$$f_{weld} := \sqrt{\left(f_b + f_a\right)^2 + f_v^{\,2}}$$ $f_{weld} = 20.46$ ksi

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

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

W_{weld} = "3/16"

NODE 114 MAX



Loads Data:

$M := 7.0$ Moment in Connection (kips-ft)

$P := 4.0$ Axial in Connection **Positive Down (Gravity)** (kips)

$V := 5.5$ Shear in Connection (kips)

Plate Data:

$B := 36.00$ Plate Width (in)

$N := 18.0$ Plate Lenght along Moment Direction (in)

$a := 4.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 10.0$ Depth of Column along Length (N) direction (in)

$b_f := 4.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate Material (Ksi)

$Fb_{plate} := 21.60$ Allowable Tensile Bending Stress of Plate Material (Ksi)
Fb=0.6*Fy .... for Steel
Fb=Fy/1.85......................for Stainless Steel

Allowable Support Stress Data:

$Fp_{supp} := 1.050$ **Allowable** compressive strength of support (ksi)
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

Concrete Bearing Stress:

$$e := \frac{12M}{\max(0.1, P)}$$

$e = 21.00$ in

$$f_b := \begin{cases} \text{"INCREASE PLATE"} & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} & \text{if } e < \frac{N}{6} \\ \frac{2P}{3 \cdot \left(\frac{N}{2} - e\right) \cdot B} & \text{if } \frac{N}{6} < e < \frac{N}{2} \\ Fp_{supp} & \text{if } e \geq \frac{N}{2} \wedge \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq Fp_{supp} \\ 0.00 & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq 0.00 \end{cases}$$

$f_b = 1.05$ ksi

Compression Length:

$$Kd := \text{if}\left[e \leq \frac{N}{6}, N, \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\frac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\frac{x^2}{3}\right) \leq M \cdot 12 \\ \quad x \leftarrow x + 0.00001 \end{array}\right]$$

$Kd = 0.40$

Tension in Bolts:

$$T_{bolts} := \begin{cases} 0.5 \cdot f_b \cdot Kd \cdot B - P \\ 0 & \text{if } e \leq \frac{N}{6} \end{cases}$$

$T_{bolts} = 3.50$ Kips

Tension per Bolt:

$$T_{Perbolts} := \frac{T_{bolts}}{6}$$

$T_{Perbolts} = 0.58$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2}$$ $m = 4.25$ in

$$n := \frac{B - 0.8 \cdot b_f}{2}$$ $n = 16.40$ in

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases}$$ $f_2 = 0.00$ ksi

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a) T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{(f_b - f_2) B \cdot m^2}{3}, (m - a) T_{bolts}\right] & \text{if } Kd > m \end{cases}$$ $M_{plate} = 30.88$ kips − in

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}}$$ $t_{plate} = 0.49$ in

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Rev1 Top MAX Connection	Date:	4/3/2026
Fastening point:			

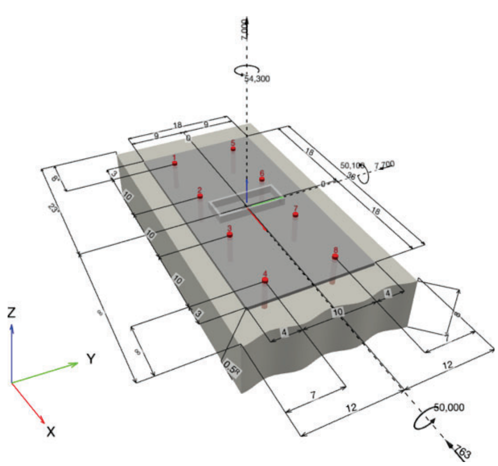
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	HILTI ∅ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 36.000 in. x 18.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X4X.375; (L x W x T) = 10.000 in. x 4.000 in. x 0.375 in.
Base material:	uncracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: present, shear: present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

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

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



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Fastening point:			

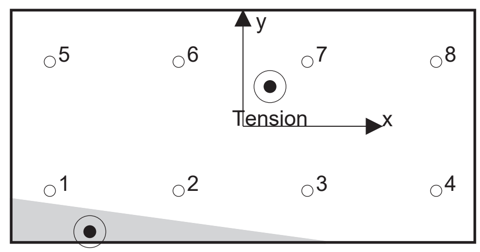
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 7,000; V_x = -763; V_y = -7,700; M_x = 50,000; M_y = -50,100; M_z = 54,300;	no	94

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	158	1,646	131	-1,641
2	358	1,196	131	-1,189
3	558	748	131	-736
4	758	312	131	-284
5	1,625	1,672	-322	-1,641
6	1,825	1,231	-322	-1,189
7	2,025	803	-322	-736
8	2,224	429	-322	-284



Max. concrete compressive strain: 0.04 [‰]
Max. concrete compressive stress: 187 [psi]
Resulting tension force in (x/y)=(2.098/3.077): 9,531 [lb]
Resulting compression force in (x/y)=(-11.895/-8.168): 2,531 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	2,224	20,808	11	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	9,531	30,478	32	OK

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

3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-3027

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

Variables

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

Calculations

N_{sa} [lb]
32,013

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
32,013	0.650	20,808	2,224

3.2 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.770	2.098	3.077	7.000	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
5.690	27	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]
880.32	127.92	0.729	0.648	1.000	1.000	12,500

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
40,637	0.750	30,478	9,531

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,672	9,996	17	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	7,738	48,012	17	OK
Concrete edge failure in direction x-**	7,807	8,964	88	OK

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

4.1 Steel Strength

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

Variables

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

Calculations

V_{sa} [lb]
16,660

Results

V_{sa} [lb]	ϕ $_{steel}$	ϕ V_{sa} [lb]	V_{ua} [lb]
16,660	0.600	9,996	1,672

4.2 Pryout Strength

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

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

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

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

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

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	6.983	0.692	7.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	5.690	27	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]
880.32	127.92	0.447	0.891	1.000	1.000	12,500

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
68,589	0.700	48,012	7,738

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.]
5.333	7.000	0.824	1.400	8.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
3.770	1.000	0.750	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]
192.00	128.00	0.907	0.962	1.000	6,522

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
11,952	0.750	8,964	7,807

*Anchor row defined by: Anchor 1, 5; Case 3 controls

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

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.313	0.871	5/3	94	OK

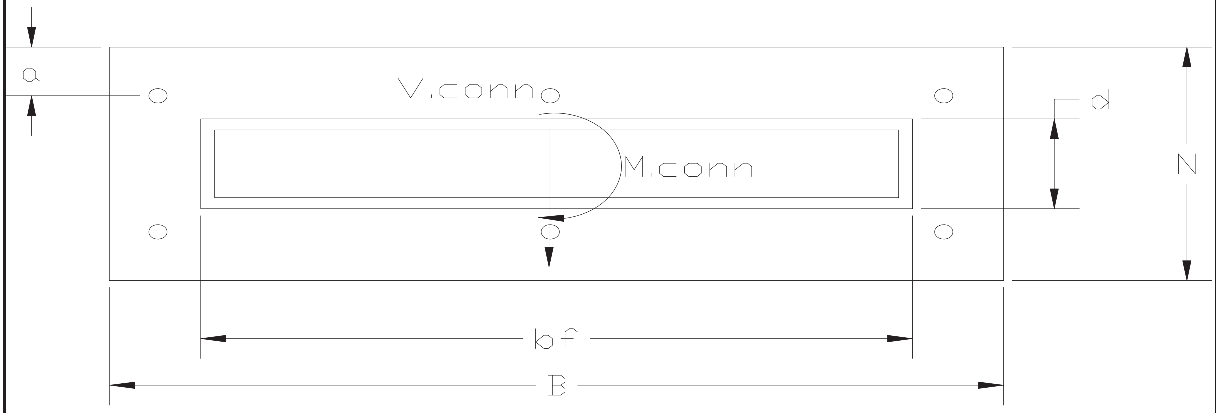
$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

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

Fastening meets the design criteria!

NODE 114 MIN



Loads Data:

$M := 6.0$ Moment in Connection (kips-ft)

$P := 7.0$ Axial in Connection **Positive Down (Gravity)** (kips)

$V := 9.0$ Shear in Connection (kips)

Plate Data:

$B := 36.00$ Plate Width (in)

$N := 18.0$ Plate Lenght along Moment Direction (in)

$a := 4.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 10.0$ Depth of Column along Length (N) direction (in)

$b_f := 4.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate Material (Ksi)

$Fb_{plate} := 21.60$ Allowable Tensile Bending Stress of Plate Material (Ksi)
Fb=0.6*Fy .... for Steel
Fb=Fy/1.85......................for Stainless Steel

Allowable Support Stress Data:

$Fp_{supp} := 1.050$ **Allowable** compressive strength of support (ksi)
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

Concrete Bearing Stress:

$$e := \frac{12M}{\max(0.1, P)}$$

$e = 10.29$ in

$$f_b := \begin{cases} \text{"INCREASE PLATE"} & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} & \text{if } e < \frac{N}{6} \\ \frac{2P}{3 \cdot \left(\frac{N}{2} - e\right) \cdot B} & \text{if } \frac{N}{6} < e < \frac{N}{2} \\ Fp_{supp} & \text{if } e \geq \frac{N}{2} \wedge \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq Fp_{supp} \\ 0.00 & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \leq 0.00 \end{cases}$$

$f_b = 1.05$ ksi

Compression Length:

$$Kd := \text{if}\left[e \leq \frac{N}{6}, N, \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\frac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\frac{x^2}{3}\right) \leq M \cdot 12 \\ \quad x \leftarrow x + 0.00001 \end{array}\right]$$

$Kd = 0.41$

Tension in Bolts:

$$T_{bolts} := \begin{cases} 0.5 \cdot f_b \cdot Kd \cdot B - P \\ 0 & \text{if } e \leq \frac{N}{6} \end{cases}$$

$T_{bolts} = 0.72$ Kips

Tension per Bolt:

$$T_{Perbolts} := \frac{T_{bolts}}{6}$$

$T_{Perbolts} = 0.12$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2}$$ $m = 4.25$ in

$$n := \frac{B - 0.8 \cdot b_f}{2}$$ $n = 16.40$ in

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases}$$ $f_2 = 0.00$ ksi

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a)T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{(f_b - f_2)B \cdot m^2}{3}, (m - a)T_{bolts}\right] & \text{if } Kd > m \end{cases}$$ $M_{plate} = 31.75$ kips − in

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}}$$ $t_{plate} = 0.49$ in

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Rev1 Top Connection	Date:	4/3/2026
Fastening point:			

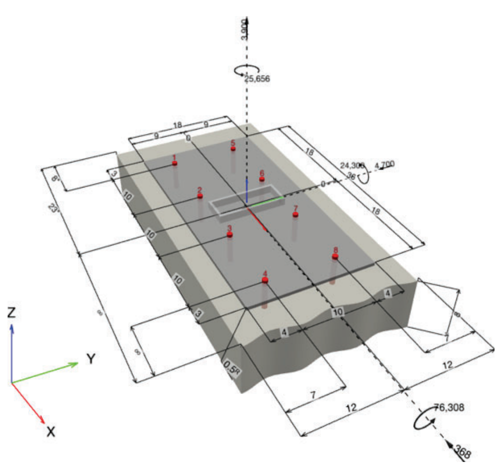
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 3/4 (5)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	HILTI ∅ 3/4 IN KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3027 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 3.770 in., h_{nom} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 36.000 in. x 18.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X4X.375; (L x W x T) = 10.000 in. x 4.000 in. x 0.375 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

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

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



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Phone I Fax:	|	E-Mail:	
Design:	Rev1 Top Connection	Date:	4/3/2026
Fastening point:			

1.1 Design results

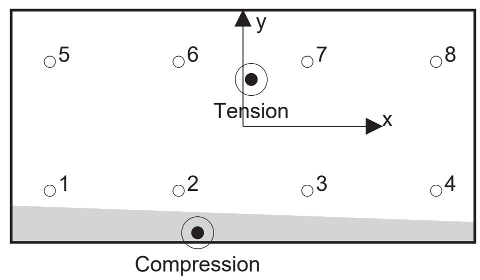
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 3,900; V_x = -368; V_y = -4,700; M_x = 76,308; M_y = -24,300; M_z = 25,656;	no	95

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	225	910	61	-908
2	284	697	61	-694
3	343	484	61	-481
4	401	274	61	-267
5	1,881	921	-153	-908
6	1,940	711	-153	-694
7	1,999	504	-153	-481
8	2,058	308	-153	-267



Max. concrete compressive strain: 0.04 [‰]
Max. concrete compressive stress: 174 [psi]
Resulting tension force in (x/y)=(0.643/3.628): 9,131 [lb]
Resulting compression force in (x/y)=(-3.523/-8.255): 5,231 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	2,058	20,808	10	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	9,131	19,257	48	OK

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

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3027

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

Variables

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

Calculations

N_{sa} [lb]
32,013

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
32,013	0.650	20,808	2,058

3.2 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.770	0.643	3.628	7.000	1.000

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

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
880.32	127.92	0.898	0.609	1.000	1.000	7,870

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
29,626	0.650	19,257	9,131

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	921	9,996	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	4,714	35,996	14	OK
Concrete edge failure in direction x-**	4,740	6,100	78	OK

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

4.1 Steel Strength

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

Variables

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

Calculations

V_{sa} [lb]
16,660

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
16,660	0.600	9,996	921

4.2 Pryout Strength

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

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

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

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

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

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.770	5.425	0.425	7.000

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

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
880.32	127.92	0.510	0.930	1.000	1.000	7,870

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
51,422	0.700	35,996	4,714

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.]
5.333	7.000	0.645	1.000	8.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
3.770	1.000	0.750	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]
192.00	128.00	0.925	0.962	1.000	6,522

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
8,714	0.700	6,100	4,740

*Anchor row defined by: Anchor 1, 5; Case 3 controls

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

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.474	0.777	5/3	95	OK

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

6 Warnings

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

Fastening meets the design criteria!

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Block Conn	Date:	4/3/2026
Fastening point:			

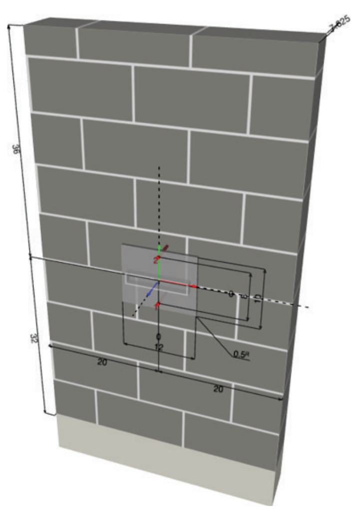
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) (Carbon Steel) 1/2 (4 1/4)**
Item number:	418073 KH-EZ 1/2"x4"
Specification text:	HILTI ∅ 1/2 (4 1/4) KWIK HUS-EZ (KH-EZ) (CARBON STEEL) WITH 3.22 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-3056 , HAMMER DRILLED INSTALLATION PER MPII
Effective embedment depth:	h_{ef} = 3.220 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3056
Issued I Valid:	10/1/2025 | 10/1/2027
Proof:	Design Method LRFD (AC01) Masonry + ACI 318-19
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 10.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X2X.250; (L x W x T) = 10.000 in. x 2.000 in. x 0.250 in.
Base material:	cracked Grout-filled CMU, f=3000, f = 2,999 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Solid Head Joint: no; open ended unit: no
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry

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

Geometry [in.]



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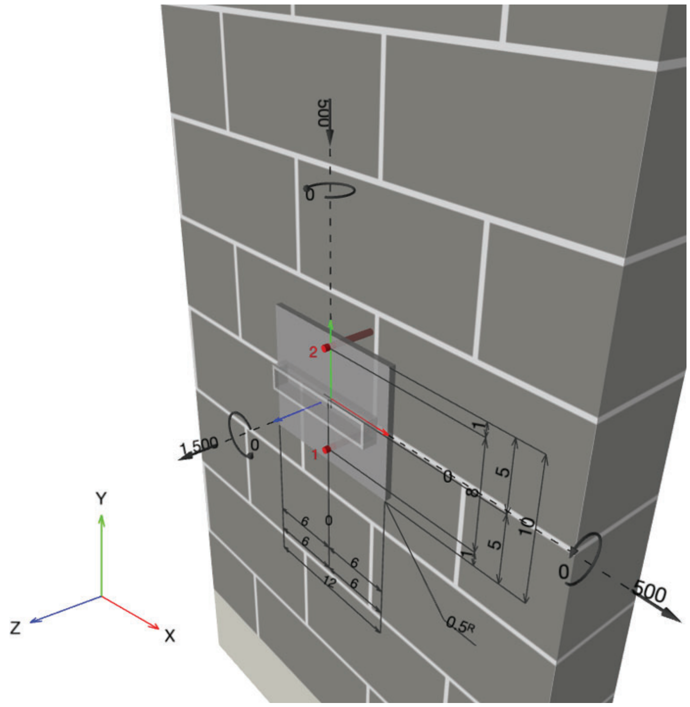
Fastening point:			

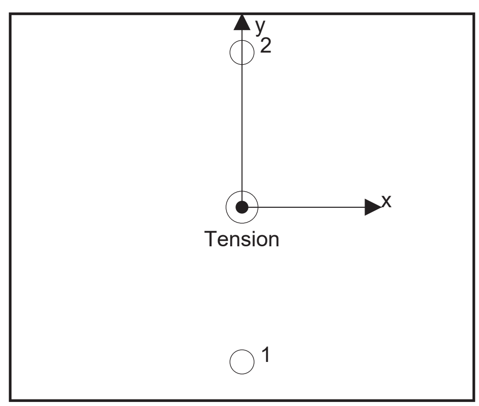
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,500; V_x = 500; V_y = -500; M_x = 0; M_y = 0; M_z = 0;	no	57

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	750	354	250	-250
2	750	354	250	-250

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(0.000/-0.000): 1,500 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

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

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	750	11,777	7	OK
Pullout	750	2,470	31	OK
Masonry breakout	1,500	3,501	43	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ESR-3056

$\phi\ N_{sa} \geq N_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

N_{sa} [lb]	ϕ	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,119	0.650	11,777	750

3.2 Pullout strength

$\phi\ N_{pn} \geq N_{ua}$ AC01 Table 3.2 + ACI 318-19 + Table 17.5.2

N_p [lb]	ϕ	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
3,800	0.650	2,470	750

3.3 Masonry breakout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec,N,m} \; \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi \; N_{mbg} \geq N_{ua}$ — AC01 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec,N,m} = \left(\frac{1}{1 + \frac{2 e_N'}{3 \cdot h_{ef}}} \right) \leq 1.00$ — ACI 318-19 Eq. (17.6.2.3.1)

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

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f_m'} \cdot h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N,m}$ [in.]	$e_{c2,N,m}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
3.220	0.000	0.000	3.813	1.000	12

λ_a	f_m' [psi]
1.000	2,999

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
141.28	93.32	1.000	1.000	0.937	3,797

Results

N_{mbg} [lb]	ϕ	$\phi \; N_{mbg}$ [lb]	N_{ua} [lb]
5,386	0.650	3,501	1,500

4 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	354	3,927	10	OK
Pryout	707	7,540	10	OK
Masonry crushing strength	354	4,102	9	OK
Masonry breakout in direction y-	707	1,407	51	OK

4.1 Steel strength

V_{sa} = ESR value   refer to ICC-ES ESR-3056

$\phi\ V_{sa} \geq V_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
6,545	0.600	3,927	354

4.2 Pryout strength

$V_{mpg} = k_{cp} \cdot N_{mbg}$

$\phi\ V_{mpg} \geq V_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

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

$A_{Nm0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c1,N}}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c2,N}}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N,m} = 0.7 + 0.3 \left(\frac{c}{c_{cr,N}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ 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.]	$\psi_{c,N}$
2	3.220	0.000	0.000	3.813	1.000

k_m	λ_a
12	1.000

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N}$	$N_{b,m}$ [lb]
141.28	93.32	1.000	1.000	0.937	3,797

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
10,772	0.700	7,540	707

4.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC01 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ AC01 Eq. (3-1)

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
2,999	0.16

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,204	0.500	4,102	354

4.4 Masonry breakout strength y-

$V_{mbg} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{ec,V,m}\ \psi_{ed,V,m}\ \psi_{c,V,m}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{mbg} \geq V_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

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

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_V}{3c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

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

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

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

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
3.220	0.500	5.083	3.813	61.00	116.28	2,999

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V,m}$	$\psi_{c,V,m}$	$\psi_{h,V,m}$	$V_{b,m}$ [lb]
0.850	1.000	0.000	1.000	1.000	1.000	4,509

Results

V_{mbg} [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
2,011	0.700	1,407	707

5 Required verifications under combined tension and shear forces

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.428	0.502	5/3	57	OK

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

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- 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!

EXTERIOR ALUMINUM CABLE RAILING DESIGN & ANALYSIS

Handrail Design:

General Data:

$L := 50.00$ Railing Span (in)

$H_{post} := 40.00$ 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

$Fv_{TR} := 5500.00$ psi

$Sx_{TR} := 0.325$ in^3

$Sy_{TR} := 0.325$ in^3

$A_{TR} := 0.736$ in^2

Cable Data:

$Fy_{cable} := 50000.00$ psi

$\phi_{cable} := 0.1875$ in

$E_{cable} := 16345000.00$ psi

Post Anchor Data:

$Fb_{PT} := 19000.00$ psi

$Fb_{PTnw} := 10600.00$ psi

$Fv_{PT} := 5500.00$ psi

$Sx_{PT} := 0.9115$ in^3

$Sy_{PT} := 0.9115$ in^3

$A_{PT} := 1.75$ in^2

$Ix_{PT} := 0.9115$ in^4

$E_{PT} := 10100.00$ ksi

Interm. Post Data:

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

$Fb_{PT.Inw} := 10600.0($ psi

$Fv_{PT.I} := 5500.00$ psi

$Sx_{PT.I} := 0.552$ in^3

$Sy_{PT.I} := 0.552$ in^3

$A_{PT.I} := 0.938$ in^2

$Ix_{PT.I} := 0.552$ in^4

$E_{PT.I} := 10100.00$ ksi

Cable Design:

$L_{cable} := 50.00$ Actual Cable Intermediate Post Span (in)

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

$L_T := 50.00$ Length of cable between anchor points (in)

$F_{ps} := 150.00$ Applied prestressing Force (lbs)

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

$$A_{cable} := \pi \cdot \frac{\phi_{cable}^2}{4}$$ $A_{cable} = 0.03$ in^2

$$I_{cable} := 0.70\left(\pi \cdot \frac{\phi_{cable}^4}{64}\right)$$ $I_{cable} = 42.47 \times 10^{-6}$ in^4

$$\Delta_V := \begin{array}{|l} x \leftarrow 0.01 \\ \text{while } \frac{4 \cdot x \cdot E_{cable} \cdot A_{cable}}{L_{cable}} \cdot \frac{\sqrt{4 \cdot x^2 + L_{cable}^2} - L_{cable}}{\sqrt{4 \cdot x^2 + L_{cable}^2} + L_T - L_{cable}} + \frac{48 \cdot E_{cable} \cdot I_{cable} \cdot x}{L_{cable}^3} + \frac{4 \cdot F_{ps} \cdot x}{L_{cable}} \le \frac{\left(50 \cdot \frac{\pi \cdot 4^2}{4 \cdot 144}\right)}{2 \cdot \cos\left(\text{asin}\left(\frac{s_{cable} + 2 \cdot x}{4 + \phi_{cable}}\right)\right)} \\ \quad x \leftarrow x + 0.0001 \end{array}$$

$\Delta_V = 0.29$ in

$$P_{ef} := \frac{\left(50 \cdot \frac{\pi \cdot 4^2}{4 \cdot 144}\right)}{2 \cos\left(\text{asin}\left(\frac{s_{cable} + 2 \cdot \Delta_V}{4 + \phi_{cable}}\right)\right)}$$ $P_{ef} = 4.18$ lbs

$$T_{cable} := \frac{P_{ef}}{2 \cdot \Delta_V} \cdot \sqrt{\Delta_V^2 + \frac{L_{cable}^2}{4}}$$ $T_{cable} = 182.79$ lbs

Cable Section Required:

$$\phi_{cable.req} := \sqrt{4 \cdot \frac{\frac{T_{cable}}{0.9 \cdot Fy_{cable}}}{\pi}}$$ $\phi_{cable.req} = 0.07$ in

Section Provided:

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

Top Railing Vertical Design:

Actual Moment for Vertical Loads:

Concentrated Load = 200 lbs

$M_{TR.200.V} := \frac{P_{200} \cdot L}{5}$ $\qquad$ $M_{TR.200.V} = 2000.00$ in − lb

Uniform Load = 50 plf

$M_{TR.50.V} := 0.1012 \cdot \left(\frac{q_{50}}{12}\right) \cdot L^2$ $\qquad$ $M_{TR.50.V} = 1054.17$ in − lb

$M_{TR.V} := \max\left(M_{TR.50.V}, M_{TR.200.V}\right)$ $\qquad$ $M_{TR.V} = 2000.00$ in − lb

Actual Shear for Vertical Loads:

Concentrated Load = 200 lbs.

$V_{TR.200} := P_{200}$ $\qquad$ $V_{TR.200} = 200.00$ lbs

Uniform Load = 50 plf.

$V_{TR.50} := 0.6 \cdot \left(\frac{q_{50}}{12} \cdot L\right)$ $\qquad$ $V_{TR.50} = 125.00$ lbs

$V_{TR.V} := \max\left(V_{TR.200}, V_{TR.50}\right)$ $\qquad$ $V_{TR.V} = 200.00$ lbs

Actual Axial for Cable Loads:

$P_{TR.V} := \frac{T_{cable} \cdot \mathrm{floor}\left(\frac{H_{post}}{s_{cable}}\right)}{2}$ $\qquad$ $P_{TR.V} = 1188.15$ 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

$$M_{TR.200.H} := \frac{P_{200} \cdot L}{5}$$ $M_{TR.200.H} = 2000.00$ in − lb

Uniform Load = 50 plf

$$M_{TR.50.H} := 0.1012 \cdot \left(\frac{q_{50}}{12}\right) \cdot L^2$$ $M_{TR.50.H} = 1.05 \times 10^3$ in − lb

For Wind Pressure

$$M_{TR.wind} := 0.1012 \cdot \left(\frac{q_{wind}}{144}\right) \cdot \left(\frac{H_{post}}{2} \cdot L^2\right)$$ $M_{TR.wind} = 351.39$ in − lb

$$M_{TR.H} := \max\left(M_{TR.200.H}, M_{TR.50.H}, M_{TR.wind}\right)$$ $M_{TR.H} = 2000.00$ 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

$$V_{TR.50.H} := 0.6 \cdot \left(\frac{q_{50}}{12}\right) \cdot L$$ $V_{TR.50.H} = 125$ lbs

For Wind Pressure

$$V_{TR.wind} := 0.6 \cdot \left(\frac{q_{wind}}{144}\right) \cdot \left(\frac{H_{post}}{2}\right) \cdot L$$ $V_{TR.wind} = 41.67$ 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:

$$P_{TR.H} := \frac{T_{cable} \cdot \text{floor}\left(\frac{H_{post}}{s_{cable}}\right)}{2}$$ $P_{TR.H} = 1188.15$ lbs.

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

Post Anchor Along X-X Axis:

Concentrated Load = 200 lbs.

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

Uniform Load = 50 plf

$R_{PT.50} := \left(\frac{q_{50}}{12}\right)\cdot L$ $R_{PT.50} = 208.33$ lbs

For Wind Pressure

$R_{PT.wind} := \left(\frac{q_{wind}}{144}\right)\cdot H_{post}\cdot L$ $R_{PT.wind} = 138.89$ lbs

$R_{PT.xx} := \max\left(R_{PT.200}, R_{PT.50}, R_{PT.wind}\right)$ $R_{PT.xx} = 208.33$ lbs

Actual Moment:

$M_{PT.xx} := 0.85R_{PT.xx}\cdot H_{post}$ $M_{PT.xx} = 7083.33$ in - lb

Actual Shear:

$V_{PT.xx} := R_{PT.xx}$ $V_{PT.xx} = 208.33$ lbs.

Section Required:

Bending Design: Section Modulus Required:

$S_{PT.xx} := \frac{M_{PT.xx}}{Fb_{PT}}$ $S_{PT.xx} = 0.37$ in^3

Shear Design: Area Required:

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

Post Anchor Section Provided:

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

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

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

Actual Moment:

$$M_{PT.yy} := \frac{T_{cable} \cdot floor\left(\frac{H_{post} - 2}{1.35 s_{cable}}\right)\left(H_{post} - 2\right)}{8}$$

$M_{PT.yy} = 7814.37$ in − lb

Actual Shear:

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

$V_{PT.yy} = 1188.15$ lbs.

Section Required:

Bending Design: Section Modulus Required:

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

$S_{PT.yy} = 0.411$ in^3

Shear Design (Area Required)

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

$A_{PT.yy} = 0.32$ in^2

Post Anchor Section Provided:

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

$BENDING_{PT.yy} = "OK"$

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

$SHEAR_{PT.yy} = "OK"$

Post Anchor Combined Stress (Bending at Hpost/2)

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

$fb_{PT.xx.0.5H} = 7771.07$ psi

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

$fb_{PT.yy} = 8573.08$ psi

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

$IE_{PT} = 0.86$

$$COMBINED_{PT} := \begin{array}{|l} "N.G." \\ "OK" \quad if \;\; 1.05 \ge IE_{PT} \end{array}$$

$COMBINED_{PT} = "OK"$

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Phone I Fax:	|	E-Mail:	
Design:	Copy - Side Conn	Date:	4/19/2023
Fastening point:			

Specifier's comments:

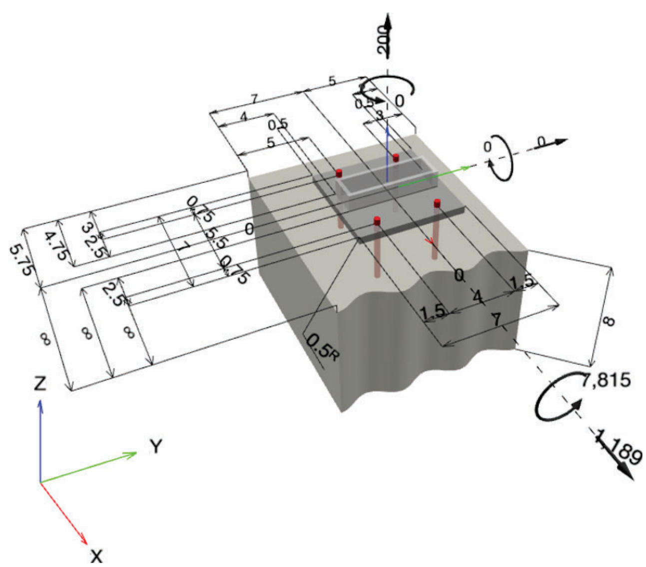
1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/8 (4 1/2)**
Item number:	418061 KH-EZ 3/8"x5"
Effective embedment depth:	$h_{ef,act}$ = 3.550 in., h_{nom} = 4.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
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 = 7.000 in. x 7.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS6X2X.250; (L x W x T) = 6.000 in. x 2.000 in. x 0.250 in.
Base material:	cracked lightweight concrete, 3000, f_c' = 3,000 psi; h = 8.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

Application also possible with KWIK-X 3/8 (3) hnom2 under the selected boundary conditions.
More information in section Alternative fastening data of this report.

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

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



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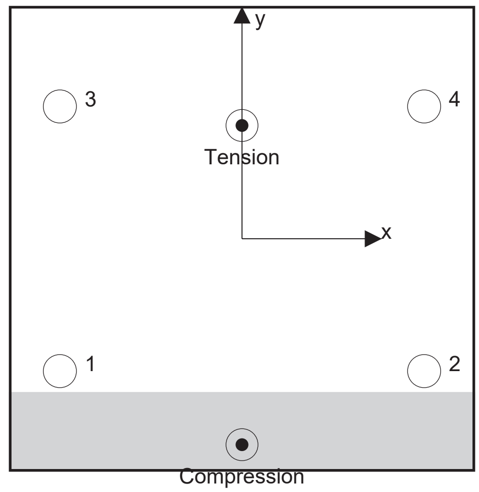
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 200; V_x = 1{,}189; V_y = 0;$ $M_x = 7{,}815; M_y = 0; M_z = 0;$	no	96

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	63	297	297	0
2	63	297	297	0
3	812	297	297	0
4	812	297	297	0

max. concrete compressive strain:	0.09 [‰]
max. concrete compressive stress:	380 [psi]
resulting tension force in (x/y)=(0.000/1.713):	1,749 [lb]
resulting compression force in (x/y)=(0.000/-3.112):	1,549 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	812	6,718	13	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,749	2,347	75	OK

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

3.1 Steel Strength

N_{sa} [lb]	ϕ	ϕN_{sa} [lb]	N_{ua} [lb]
10,335	0.650	6,718	812

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3.2 Concrete Breakout Failure

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	c_{ac} [in.]	$\psi_{c,N}$	h_{ef} [in.]
162.00	100.00	3.000	14.200	1.000	3.333

$e_{c1,N}$ [in.]	$\psi_{ec1,N}$	$e_{c2,N}$ [in.]	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	k_{cr}
0.000	1.000	1.713	0.745	0.880	1.000	17

λ_a	N_b [lb]	ϕ	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
0.600	3,400	0.650	2,347	1,749

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	297	3,111	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1,189	6,786	18	OK
Concrete edge failure in direction y+**	1,189	2,244	53	OK

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

4.1 Steel Strength

V_{sa} [lb]	ϕ	ϕ V_{sa} [lb]	V_{ua} [lb]
5,185	0.600	3,111	297

4.2 Pryout Strength

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	k_{cp}	c_{ac} [in.]	$\psi_{c,N}$	h_{ef} [in.]
162.00	100.00	3.000	2	14.200	1.000	3.333

$e_{c1,V}$ [in.]	$\psi_{ec1,V}$	$e_{c2,V}$ [in.]	$\psi_{ec2,V}$	$\psi_{ed,N}$	$\psi_{cp,N}$	k_{cr}
0.000	1.000	0.000	1.000	0.880	1.000	17

λ_a	N_b [lb]	ϕ	ϕ V_{cpg} [lb]	V_{ua} [lb]
0.600	3,400	0.700	6,786	1,189

4.3 Concrete edge failure in direction y+

l_e [in.]	d_a [in.]	c_{a1} [in.]	A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]
3.000	0.375	3.000	58.50	40.50

$\psi_{ed,V}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V}$	$\psi_{c,V}$	$\psi_{h,V}$
1.000	2.000	0.000	1.000	1.000	1.000

λ_a	V_b [lb]	ϕ	ϕ V_{cbg} [lb]	V_{ua} [lb]
0.600	1,109	0.700	2,244	1,189

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.745	0.530	5/3	96	OK

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

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!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

EXTERIOR GLASS RAILING DESIGN & ANALYSIS

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

Wind Velocity (mph) as per FBC-2023 Section 1620.2

	Miami-Dade	Broward
Risk Category I	V=165 mph	V=156 mph
Risk Category II	V=175 mph	V=170 mph
Risk Category III	V=186 mph	V=180 mph
Risk Category IV	V=195 mph	V=185 mph

General Wind Data:

$V := 175.00$ Wind Velocity (mph)

$K_{zt} := 1.00$ Topographic Factor ASCE7-22 Sections 26.8.1 & 26.8.2

$K_d := 0.85$ Wind Directionality Factor ASCE 7-22 Section 26.6

$G := 0.85$ Gust Factor (Rigid Structure) ASCE 7-22 Section 26.11

$K_e := 1.00$ Ground Elevation Factor ASCE 7-22 Section 26.9

$C_f := 1.80$ Net Force Coefficients (see Figure 6-20 through 6-23)

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

$\alpha := 9.8$

$zg := 2460$

Values for Terrain exposure constants α and zg:
Exposure B------ Value α =7.5 , Value zg=3280
Exposure C------ Value α =9.8 , Value zg=2460
Exposure D------ Value α =11.5 , Value zg=1935

General Sign Data:

$Z := 18.0$ Height of Top of Sign (ft)

Then

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

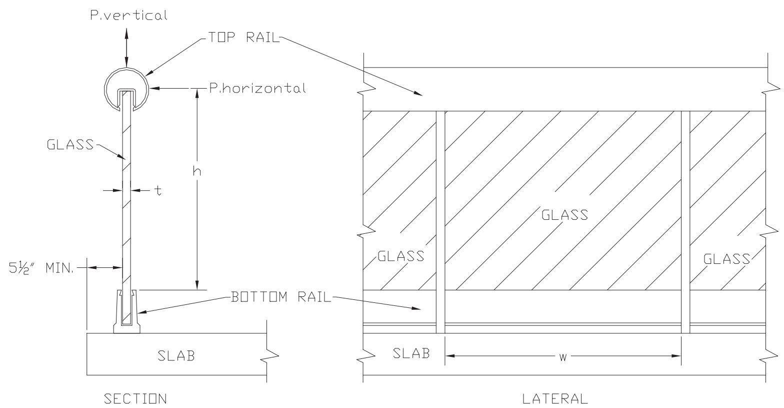
$$K_z := 2.41\left(\frac{Z}{zg}\right)^{\frac{2}{\alpha}}$$ $K_z = 0.88$

$$q_z := 0.00256 K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2$$ $q_z = 58.87$ psf

Allowable Design Wind Loads:

$$p_z := \max\left[0.6\left(q_z \cdot G \cdot C_f\right), 10\right]$$ $p_z = 54.04$ psf

Glass Railing Design



Loads Data:

$P_{200} := 200.00$	Concentrated Load (lbs)
$q_{50} := 50.00$	Uniform Load (plf)
$q_{wind} := 54.04$	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:

$h := 37.00$	Height of Glass Lite Pannel in Cantilever (in)
$t := 0.75$	Thickness of Glass Pannel (in)
$w := 48.00$	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

$L := \begin{cases} h \\ w \text{ if } w < h \end{cases}$ $L = 37.00$ in

$S_x := \frac{L \cdot t^2}{6}$ $S_x = 3.47$ in^3

$I_x := \frac{L \cdot t^3}{12}$ $I_x = 1.30$ in^4

$A := L \cdot t$ $A = 27.75$ in^2

Maximum Glass Moment in Length (L):

Concentrated Load = 200 lbs.

$M_{200} := P_{200} \cdot h$ $M_{200} = 7400.00$ lbs − in

Uniform Load = 50 plf

$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$ $M_{50} = 5704.17$ lbs − in

Uniform Distributed Wind Load

$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$ $M_{wind} = 9504.47$ lbs − in

$M_{max} := \max(M_{200}, M_{50}, M_{wind})$ $M_{max} = 9504.47$ lbs − in

Maximum Glass Shear in Length (L):

Concentrated Load = 200 lbs.

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

Uniform Load = 50 plf

$V_{50} := q_{50} \cdot \frac{L}{12}$ | $V_{50} = 154.17$ lbs

Uniform Distributed Wind Load

$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$ | $V_{wind} = 513.76$ lbs

$V_{max} := \max(V_{200}, V_{50}, V_{wind})$ | $V_{max} = 513.76$ lbs

Section Required:

Bending Design:
Section Modulus Required $Sx_r := \frac{M_{max}}{F_b}$ | $Sx_r = 1.58$ in^3

Shear Design:
Area Required $A_r := \frac{V_{max}}{F_v}$ | $A_r = 0.17$ in^2

Section Provided:

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

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

Check Deflection:

$\Delta_{permissible} := \frac{h}{30}$ | $\Delta_{permissible} = 1.23$ in

$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_x}$ | $\Delta_{200} = 0.25$ in

$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_x}$ | $\Delta_{wind} = 0.24$ in

$DEFLECTION_{glass} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} & \text{if } \max(\Delta_{200}, \Delta_{wind}) \leq \Delta_{permissible} \end{cases}$ | $DEFLECTION_{glass} = \text{"O.K."}$

Base Shoe Connection Design:

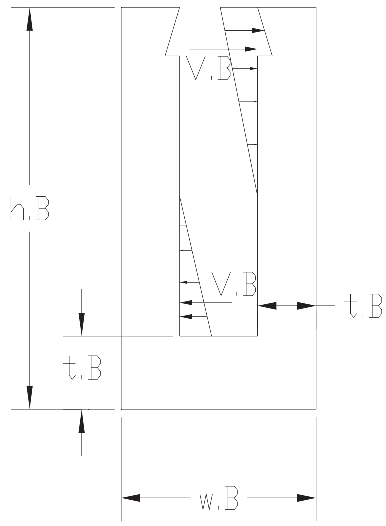
$h_B := 4.92$ Height of Bottom Rail (in)

$t_B := 0.85$ Thick of Wall in Bottom Rail (in)

$w_B := 1.88$ Width of Bottom Rail (in)

$F_{b.} := 10000.00$ Allowable Bending Stress in Bottom Rail (psi)

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



Alloy 6060-T6

$$L_1 := \begin{vmatrix} h + h_B \\ w \ \text{ if } \ w < h + h_B \end{vmatrix}$$

$L_1 = 41.92$ in

Maximum Mounting Moment in Length (L1):

Concentrated Load = 200 lbs.

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

$M_{B200} = 8384.00$ lbs − in

Uniform Load = 50 plf

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

$M_{B50} = 7322.03$ lbs − in

Uniform Distributed Wind Load

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

$M_{Bwind} = 13822.50$ lbs − in

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

$M_{Bmax} = 13822.50$ lbs − in

Maximum Mounting Shear in Length (L1):

Conc. Load = 200 lbs.

$$V_{B200} := \frac{M_{B200}}{\frac{4}{6}\cdot(h_B - t_B)} + P_{200} \qquad V_{B200} = 3289.93 \text{ lbs}$$

Uniform Load = 50 plf

$$V_{B50} := \frac{M_{B50}}{\frac{4}{6}\cdot(h_B - t_B)} + q_{50}\cdot\frac{L_1}{12} \qquad V_{B50} = 2873.20 \text{ lbs}$$

Uniform Dist. Wind Load

$$V_{Bwind} := \frac{M_{Bwind}}{\frac{4}{6}\cdot(h_B - t_B)} + \frac{q_{wind}}{144}\cdot L_1\cdot(h + h_B) \qquad V_{Bwind} = 5753.76 \text{ lbs}$$

$$V_{Bmax} := \max(V_{B200}, V_{B50}, V_{Bwind}) \qquad V_{Bmax} = 5753.76 \text{ lbs}$$

Check Resisting Couple in the Wall`s Molding:

$$F_B := \frac{M_{Bmax}}{h_B} \qquad F_B = 2809.45 \text{ lbs}$$

Geometric Inertia & Area in Wall`s Molding:

$$S_B := \frac{L_1\cdot t_B^2}{6} \qquad S_B = 5.05 \text{ in}^3$$

$$A_B := L_1\cdot t_B \qquad A_B = 35.63 \text{ in}^2$$

Check Bending Stress in Molding Wall:

$$f_b := \frac{M_{Bmax}}{S_B} \qquad f_b = 2738.29 \text{ psi}$$

BENDING := | "N.G."
"OK" if $f_b \le F_{b.}$

BENDING = "OK"

Check Shear Stress in Molding Wall:

$$f_v := \frac{F_B}{A_B} \qquad f_v = 78.85 \text{ psi}$$

SHEAR := | "N.G."
"OK" if $f_v \le F_{v.}$

SHEAR = "OK"

Check Combined Bending/Shear in Wall`s Monting:

$$C_{BV} := \left(\frac{f_b}{F_{b.}}\right) + \left(\frac{f_v}{F_{v.}}\right) \qquad C_{BV} = 0.29$$

COMBINED := | "N.G."
"OK" if $C_{BV} \le 1.00$

COMBINED = "OK"

ANCHOR AT BALCONY

Table 2 - Hilti KWIK HUS-EZ and KWIK HUS-EZ P design Strength with concrete / pullout failure in uncracked concrete[1,2,3,4]

Nominal anchor diameter in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
1/4 (6.4)	1-5/8 (41)	585 (2.6)	620 (2.8)	675 (3.0)	765 (3.4)	1,075 (4.8)	1,180 (5.2)	1,360 (6.0)	1,670 (7.4)
	2-1/2 (64)	1,525 (6.8)	1,670 (7.4)	1,930 (8.6)	2,365 (10.5)	2,235 (9.9)	2,450 (10.9)	2,825 (12.6)	3,460 (15.4)
3/8 (9.5)	1-5/8 (41)	910 (4.0)	1,000 (4.4)	1,155 (5.1)	1,415 (6.3)	980 (4.4)	1,075 (4.8)	1,245 (5.5)	1,520 (6.8)
	2-1/8 (54)	1,490 (6.6)	1,635 (7.3)	1,885 (8.4)	2,310 (10.3)	1,605 (7.1)	1,760 (7.8)	2,030 (9.0)	2,485 (11.1)
	2-1/2 (64)	1,980 (8.8)	2,165 (9.6)	2,505 (11.1)	3,065 (13.6)	2,130 (9.5)	2,335 (10.4)	2,695 (12.0)	3,300 (14.7)
	3-1/4 (83)	3,085 (13.7)	3,375 (15.0)	3,900 (17.3)	4,775 (21.2)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
1/2 (12.7)	2-1/4 (57)	1,645 (7.3)	1,800 (8.0)	2,080 (9.3)	2,550 (11.3)	1,770 (7.9)	1,940 (8.6)	2,240 (10.0)	2,745 (12.2)
	3 (76)	2,785 (12.4)	3,050 (13.6)	3,525 (15.7)	4,315 (19.2)	3,000 (13.3)	3,285 (14.6)	3,795 (16.9)	4,645 (20.7)
	4-1/4 (108)	5,070 (22.6)	5,555 (24.7)	6,415 (28.5)	7,855 (34.9)	10,920 (48.6)	11,965 (53.2)	13,815 (61.5)	16,920 (75.3)
5/8 (15.9)	3-1/4 (83)	3,240 (14.4)	3,550 (15.8)	4,100 (18.2)	5,025 (22.4)	3,490 (15.5)	3,825 (17.0)	4,415 (19.6)	5,410 (24.1)
	5 (127)	6,705 (29.8)	7,345 (32.7)	8,485 (37.7)	10,390 (46.2)	14,445 (64.3)	15,825 (70.4)	18,270 (81.3)	22,380 (99.6)
3/4 (19.1)	4 (102)	4,380 (19.5)	4,795 (21.3)	5,540 (24.6)	6,785 (30.2)	9,430 (41.9)	10,330 (45.9)	11,930 (53.1)	14,610 (65.0)
	6-1/4 (159)	9,345 (41.6)	10,235 (45.5)	11,820 (52.6)	14,475 (64.4)	20,125 (89.5)	22,045 (98.1)	25,455 (113.2)	31,175 (138.7)

Table 3 - Hilti KWIK HUS-EZ and KWIK HUS-EZ P design Strength with concrete / pullout failure in cracked concrete[1,2,3,4,5]

Nominal anchor diameter in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
1/4 (6.4)	1-5/8 (41)	300 (1.3)	315 (1.4)	345 (1.5)	390 (1.7)	765 (3.4)	835 (3.7)	965 (4.3)	1,180 (5.2)
	2-1/2 (64)	760 (3.4)	830 (3.7)	960 (4.3)	1,175 (5.2)	1,585 (7.1)	1,735 (7.7)	2,000 (8.9)	2,450 (10.9)
3/8 (9.5)	1-5/8 (41)	475 (2.1)	520 (2.3)	600 (2.7)	730 (3.2)	695 (3.1)	760 (3.4)	880 (3.9)	1,080 (4.8)
	2-1/8 (54)	1,055 (4.7)	1,155 (5.1)	1,335 (5.9)	1,635 (7.3)	1,135 (5.0)	1,245 (5.5)	1,440 (6.4)	1,760 (7.8)
	2-1/2 (64)	1,400 (6.2)	1,535 (6.8)	1,775 (7.9)	2,170 (9.7)	1,510 (6.7)	1,655 (7.4)	1,910 (8.5)	2,340 (10.4)
	3-1/4 (83)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2 (12.7)	2-1/4 (57)	1,035 (4.6)	1,135 (5.0)	1,310 (5.8)	1,605 (7.1)	1,115 (5.0)	1,220 (5.4)	1,410 (6.3)	1,725 (7.7)
	3 (76)	1,755 (7.8)	1,920 (8.5)	2,220 (9.9)	2,715 (12.1)	1,890 (8.4)	2,070 (9.2)	2,390 (10.6)	2,925 (13.0)
	4-1/4 (108)	3,190 (14.2)	3,495 (15.5)	4,040 (18.0)	4,945 (22.0)	6,875 (30.6)	7,530 (33.5)	8,695 (38.7)	10,650 (47.4)
5/8 (15.9)	3-1/4 (83)	2,040 (9.1)	2,235 (9.9)	2,580 (11.5)	3,165 (14.1)	2,200 (9.8)	2,410 (10.7)	2,780 (12.4)	3,405 (15.1)
	5 (127)	4,225 (18.8)	4,625 (20.6)	5,340 (23.8)	6,540 (29.1)	9,095 (40.5)	9,965 (44.3)	11,505 (51.2)	14,090 (62.7)
3/4 (19.1)	4 (102)	2,755 (12.3)	3,020 (13.4)	3,485 (15.5)	4,270 (19.0)	5,940 (26.4)	6,505 (28.9)	7,510 (33.4)	9,200 (40.9)
	6-1/4 (159)	5,885 (26.2)	6,445 (28.7)	7,440 (33.1)	9,115 (40.5)	12,670 (56.4)	13,880 (61.7)	16,030 (71.3)	19,630 (87.3)

1 1 See Section 3.1.9 to convert design strength value to ASD value.
2 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.
3 Apply spacing, edge distance, and concrete thickness factors in Tables 6 through 15 as necessary. Compare to the steel values in Table 4. The lesser of the values is to be used for the design.
4 Tabular values are for normal weight concrete only. For lightweight concrete multiply design strength by λa as follows: For sand-lightweight, λa = 0.68. For all-lightweight, λa = 0.60.
5 Tabular values are for static loads only. For seismic tension loads, multiply cracked concrete tabular values in tension by the following reduction factors:
1/4-in diameter by 1-5/8-in nominal embedment depth - $a_{N,seis}$ = 0.60
All other sizes - $a_{N,seis}$ = 0.75
No reduction needed for seismic shear. See Section 3.1.9 for additional information on seismic applications.

Table 8 - Load Adjustment Factors for 3/8-in. Diameter KWIK HUS-EZ and KWIK HUS-EZ P in Uncracked Concrete[1,4]

3/8-in. KH-EZ uncracked concrete	Spacing factor in tension f_{AN}				Edge distance factor in tension f_{RN}				Spacing factor in shear[3] f_{AV}				Edge distance in shear: ⊥ toward edge f_{RV}				Edge distance in shear: II to and away from edge f_{RV}				Conc. thickness factor in shear[4] f_{HV}			
Embedment h_{nom} in.	1-5/8	2-1/8	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4
(mm)	(41)	(54)	(64)	(83)	(41)	(54)	(64)	(83)	(41)	(54)	(64)	(83)	(41)	(54)	(64)	(83)	(41)	(54)	(64)	(83)	(41)	(54)	(64)	(83)
Spacing (s)/edge distance (c_a)/concrete thickness (h) - in. (mm)																								
1-1/2 (38)	n/a	n/a	n/a	n/a	0.58	0.62	0.63	0.57	n/a	n/a	n/a	n/a	0.49	0.32	0.25	0.08	0.58	0.62	0.50	0.17	n/a	n/a	n/a	n/a
2 (51)	n/a	n/a	n/a	n/a	0.76	0.75	0.75	0.66	n/a	n/a	n/a	n/a	0.75	0.49	0.38	0.13	0.76	0.75	0.75	0.26	n/a	n/a	n/a	n/a
2-1/4 (57)	0.84	0.74	0.70	0.65	0.86	0.82	0.81	0.70	0.65	0.62	0.60	0.55	0.90	0.59	0.46	0.16	0.90	0.82	0.81	0.31	n/a	n/a	n/a	n/a
2-1/2 (64)	0.88	0.77	0.72	0.67	0.95	0.91	0.88	0.75	0.67	0.63	0.61	0.55	1.00	0.69	0.54	0.18	1.00	0.91	0.88	0.37	n/a	n/a	n/a	n/a
3 (76)	0.95	0.82	0.77	0.70	1.00	1.00	1.00	0.85	0.71	0.66	0.63	0.56		0.90	0.71	0.24		1.00	1.00	0.48	n/a	n/a	n/a	n/a
3-1/4 (83)	0.99	0.85	0.79	0.72				0.90	0.72	0.67	0.64	0.57		1.00	0.80	0.27				0.54	0.95	n/a	n/a	n/a
3-1/2 (89)	1.00	0.88	0.81	0.73				0.95	0.74	0.68	0.65	0.58			0.89	0.30				0.61	0.98	n/a	n/a	n/a
4 (102)		0.93	0.86	0.77				1.00	0.78	0.71	0.68	0.59			1.00	0.37				0.74	1.00	0.91	0.84	n/a
4-1/2 (114)		0.99	0.90	0.80					0.81	0.73	0.70	0.60				0.44				0.88			0.89	n/a
4-3/4 (121)		1.00	0.93	0.82					0.83	0.75	0.71	0.60				0.48				0.96			0.91	0.639
5 (127)			0.95	0.83					0.84	0.76	0.72	0.61				0.52				1.00			0.94	0.655
6 (152)			1.00	0.90					0.91	0.81	0.76	0.63				0.68							1.00	0.718
7 (178)				0.97					0.98	0.86	0.81	0.65				0.86								0.775
8 (203)				1.00					1.00	0.91	0.85	0.67				1.00								0.829
9 (229)										0.97	0.90	0.69												0.879
10 (254)										1.00	0.94	0.71												0.927
11 (279)											0.98	0.74												0.972
12 (305)											1.00	0.76												1.000
14 (356)																								
16 (406)																								
18 (457)																								
20 (508)																								
24 (610)																								

8" spacing 4" edge distance
T= 4337.5 x 1.00 x 1.00 = 4337.5 lbs
V= 9345 x 0.67 x 0.37 = 2082.36 lbs
.
Ta= 4337.5 lbs/1.44 = 3012.15 lbs
Va= 2316.62 lbs/1.44 = 1608.77 lbs
.
Okay for concrete connection at balcony

Table 9 - Load Adjustme… d Concrete[1,4]

3/8-in. KH-EZ cracked concrete	Spacing in tens… f_{AN}																… d away … edge f_{RV}				Conc. thickness factor in shear[4] f_{HV}			
Embedment h_{nom} in.	1-5/8	2-1/8																	2-1/2	3-1/4	1-5/8	2-1/8	2-1/2	3-1/4
(mm)	(41)	(54)																	(64)	(83)	(41)	(54)	(64)	(83)
Spacing (s)/edge distance (c_a)/concrete thickness (h) - in. (mm)																								
1-1/2 (38)	n/a	n/a	n/a	n/a	0.92	0.74	0.66	0.57	n/a	n/a	n/a	n/a	0.49	0.32	0.25	0.09	0.92	0.64	0.50	0.17	n/a	n/a	n/a	n/a
2 (51)	n/a	n/a	n/a	n/a	1.00	0.90	0.79	0.66	n/a	n/a	n/a	n/a	0.76	0.50	0.39	0.13	1.00	0.90	0.77	0.26	n/a	n/a	n/a	n/a
2-1/4 (57)	0.84	0.74	0.70	0.65	1.00	0.98	0.85	0.70	0.66	0.62	0.60	0.55	0.90	0.59	0.46	0.16	1.00	0.98	0.85	0.31	n/a	n/a	n/a	n/a
2-1/2 (64)	0.88	0.77	0.72	0.67	1.00	1.00	0.92	0.75	0.67	0.63	0.61	0.55	1.00	0.69	0.54	0.18	1.00	1.00	0.92	0.37	n/a	n/a	n/a	n/a
3 (76)	0.95	0.82	0.77	0.70	1.00		1.00	0.85	0.71	0.66	0.63	0.56	1.00	0.91	0.71	0.24	1.00	1.00	1.00	0.48	n/a	n/a	n/a	n/a
3-1/4 (83)	0.99	0.85	0.79	0.72				0.90	0.73	0.67	0.64	0.57		1.00	0.80	0.27				0.55	0.95	n/a	n/a	n/a
3-1/2 (89)	1.00	0.88	0.81	0.73				0.95	0.74	0.68	0.65	0.58			0.90	0.31				0.61	0.98	n/a	n/a	n/a
4 (102)		0.93	0.86	0.77				1.00	0.78	0.71	0.68	0.59			1.00	0.37				0.75	1.00	0.91	0.84	n/a
4-1/2 (114)		0.99	0.90	0.80					0.81	0.73	0.70	0.60				0.44				0.89		0.97	0.89	n/a
4-3/4 (121)		1.00	0.93	0.82					0.83	0.75	0.71	0.60				0.48				0.97		1.00	0.92	0.64
5 (127)			0.95	0.83					0.85	0.76	0.72	0.61				0.52				1.00			0.94	0.66
6 (152)			1.00	0.90					0.92	0.81	0.77	0.63				0.69							1.00	0.72
7 (178)				0.97					0.98	0.87	0.81	0.65				0.86								0.78
8 (203)				1.00					1.00	0.92	0.85	0.67				1.00								0.83
9 (229)										0.97	0.90	0.69												0.88
10 (254)										1.00	0.94	0.72												0.93
11 (279)											0.99	0.74												0.97
12 (305)											1.00	0.76												1.00
14 (356)												0.80												
16 (406)												0.85												
18 (457)												0.89												
20 (508)												0.93												
24 (610)												1.00												

1 Linear interpolation not permitted.
2 When combining multiple load adjustment factors (e.g. for a 4 anchor pattern in a corner with thin concrete member) the design can become very conservative. To optimize the design, use Hilti PROFIS Anchor Design software or perform anchor calculation using design equations from ACI 318-14 Chapter 17.
3 Spacing factor reduction in shear, f_{AV}, assumes an influence of a nearby edge. If no edge exists, then $f_{AV} = f_{AN}$.
4 Concrete thickness reduction factor in shear, f_{HV}, assumes an influence of a nearby edge. If no edge exists, then f_{HV} = 1.0.

If a reduction factor value is in a shaded cell, this indicates that this specific edge distance may not be permitted with a certain spacing (or vice versa). Check table 5 and figure 2 of this section to calculate permissable edge distance, spacing and concrete thickness combinations.

Check Anchors:

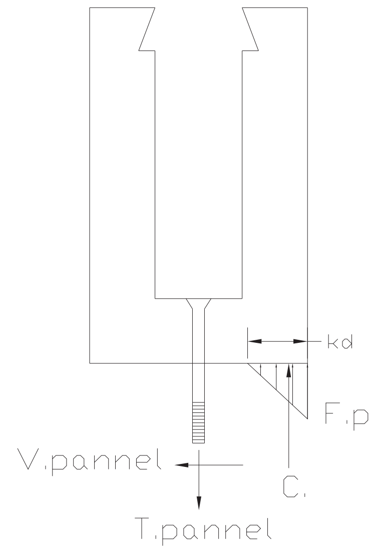
$s_{bolt} := 8.00$ Spacing of Bolts Per Pannels

$T_{bolt} := 3012.15$ Allowable Tension Per Bolt (lbs)

$V_{bolt} := 1608.77$ Allowable Shear Per Bolt (lbs)

$F_p := 5000.00$ Allowable Compressive Strength in Support (psi)

3/8" HILTI KH-EZ S anchor W/ 3-1/4" min embed, 4" min edge distance and 8" spaced o.c max



Length of Compression Zone:

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

$kd = 0.15$ in

Tensile Load on Cap Screws Per Width of Glass Pannel:

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

$T_{pannel} = 15519.80$ lbs

Shear Load on Cap Screws Per Width of Glass Pannel:

$V_{pannel} := V_{max}$

$V_{pannel} = 513.76$ lbs

Check Compressive Stress in Support:

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

$f_p = 5000.00$ psi

COMPRESSION := | "N.G."
| "OK" if $F_p \ge f_p$

COMPRESSION = "OK"

Number of Anchors Per Loaded Triangle:

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

$N_{anchors} = 6.00$ bolts

Check Tension in Anchors:

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

$T_d = 2586.63$ lbs

$$TENSION := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } T_{bolt} \geq T_d \end{vmatrix}$$

TENSION = "OK"

Check Shear in Anchors:

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

$V_d = 98.04$ lbs

$$SHEAR := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } V_{bolt} \geq V_d \end{vmatrix}$$

SHEAR = "OK"

Check Combined Tension/Shear in Anchors:

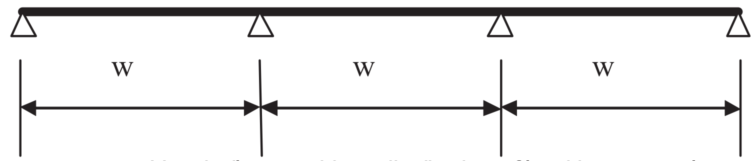
$$C_{TV} := \left(\frac{T_d}{T_{bolt}}\right)^{1.3} + \left(\frac{V_d}{V_{bolt}}\right)^{1.3}$$

$C_{TV} = 0.85$

$$COMBINED_1 := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } C_{TV} \leq 1.00 \end{vmatrix}$$

$COMBINED_1$ = "OK"

Top Railing Design:



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

$F_{b.top} := 18000.00$	Top Railing Allowable Bending Stress (psi)
$F_{v.top} := 12000.00$	Top Railing Allowable Shear Stress (psi)
$Sx_{top} := 0.1391$	Top Railing Inertia Modulus for Vertical Loads (in^3)
$Sy_{top} := 0.2124$	Top Railing Inertia Modulus for Horizontal Loads (in^3)
$A_{top} := 0.5110$	Top Railing Area (in^2)

Maximum Moment:

Concentrated Load = 200 lbs. $M_{200.top} := \frac{P_{200} \cdot w}{5}$ $M_{200.top} = 1920.00$ $lb - in$

Uniform Load = 50 plf $M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot w^2$ $M_{50.top} = 971.52$ $lb - in$

$M_{max.top} := \max(M_{200.top}, M_{50.top})$ $M_{max.top} = 1920.00$ $lb - in$

Maximum Shear:

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

Uniform Load = 50 plf $V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot w$ $V_{50.top} = 120.00$ lbs

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

Section Required:

Bending Design:
Section Modulus Required $Shr := \frac{M_{max.top}}{F_{b.top}}$ $Shr = 0.11$ in^3

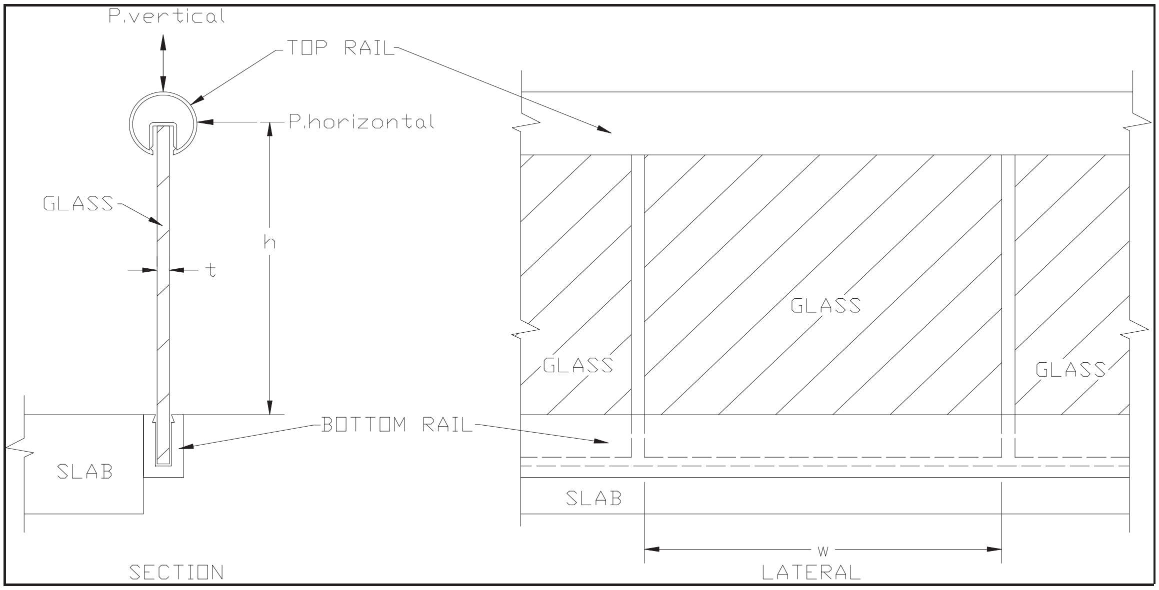
Shear Design:
Area Required $Ahr := \frac{1.5 \cdot V_{max.top}}{F_{v.top}}$ $Ahr = 0.03$ in^2

Section Provided:

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

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

INTERIOR GLASS RAILING SIDE MOUNTED BASE SHOE DESIGN & ANALYSIS



Loads Data:

$P_{200} := 200.00$	Concentrated Load (lbs)
$q_{50} := 50.00$	Uniform Load (plf)
$q_{wind} := 10.00$	Uniform Distributed Load (psf)

Glass Data:

$E := 10400000.00$	Modulus of Elasticity of Glass (psi)
$Mr_{flexure} := 24000.00$	Modulus of Rupture of Glass in Flexure (psi)
$Mr_{shear} := 120000.00$	Modulus of Rigity of glass in Shear (psi)
$SF := 4.00$	Safety Factor

Geometric Glass Railing Data:

$h := 45.00$	Height of Glass Pannel in Cantilever (in)
$t := 0.5$	Thickness of Glass Pannel (in)
$w := 48.00$	Width of Glass Pannel (in)

Then

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

$F_v := \frac{Mr_{shear}}{SF}$ $F_v = 30000.00$ psi

$L := \begin{cases} h \\ w \text{ if } w < h \end{cases}$ $L = 45.00$ in

$S_x := \frac{L \cdot t^2}{6}$ $S_x = 1.88$ in^3

$I_x := \frac{L \cdot t^3}{12}$ $I_x = 0.47$ in^4

$A := L \cdot t$ $A = 22.50$ in^2

Actual Glass Moment in Length (L):

Concentrated Load = 200 lbs.

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

Uniform Load = 50 plf

$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$ $M_{50} = 8437.50$ lbs − in

Uniform Distributed Wind Load

$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$ $M_{wind} = 3164.06$ lbs − in

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

Actual Glass Shear in Length (L):

Concentrated Load = 200 lbs.

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

Uniform Load = 50 plf

$V_{50} := q_{50} \cdot \frac{L}{12}$ $V_{50} = 187.50$ lbs

Uniform Distributed Wind Load

$$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$$ $V_{wind} = 140.63$ lbs

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

Section Required:

Bending Design:
Section Modulus Required

$$Sx_r := \frac{M_{200}}{F_b}$$ $Sx_r = 1.5$ in^3

Shear Design:
Area Required

$$A_r := \frac{V_{max}}{F_v}$$ $A_r = 0.01$ in^2

Section Provided:

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

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

Check Deflection:

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

$$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_x}$$ $\Delta_{200} = 1.25$ in

$$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_x}$$ $\Delta_{wind} = 0.33$ in

$$\Delta_{max} := \max(\Delta_{wind}, \Delta_{200})$$ $\Delta_{max} = 1.25$ in

$$DEFLECTION_{glass} := \begin{vmatrix} \text{"O.K."} \\ \text{"N.G."} \quad \text{if } \Delta_{permissible} \leq \Delta_{max} \end{vmatrix}$$ $DEFLECTION_{glass} = \text{"O.K."}$

Base Shoe Connection Design:

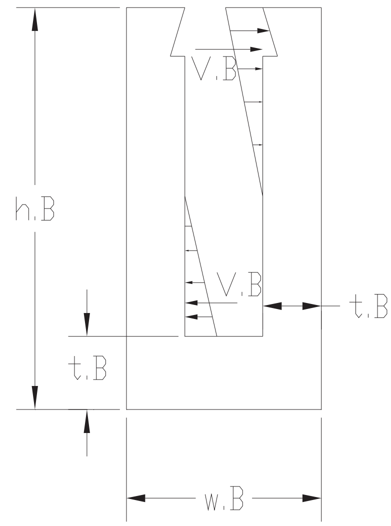
$h_B := 4.92$ Height of Bottom Rail (in)

$t_B := 0.85$ Thick of Wall in Bottom Rail (in)

$w_B := 1.88$ Width of Bottom Rail (in)

$F_{b.} := 10000.00$ Allowable Bending Stress in Bottom Rail (psi)

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



Alloy 6063-T5

$$L_1 := \begin{vmatrix} h + \frac{h_B}{2} \\ w \text{ if } w < h + h_B \end{vmatrix}$$

$L_1 = 48.00$ in

Actual Mounting Moment in Length (L1):

Concentrated Load = 200 lbs.

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

$M_{B200} = 9984.00$ lbs − in

Uniform Load = 50 plf

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

$M_{B50} = 9984.00$ lbs − in

Uniform Distributed Wind Load

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

$M_{Bwind} = 4153.34$ lbs − in

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

$M_{Bmax} = 9984.00$ lbs − in

Actual Mounting Shear in Length (L1):

Concentrated Load = 200 lbs.

$$V_{B200} := \frac{M_{B200}}{\frac{4}{6}\cdot(h_B - t_B)} + P_{200}$$

$V_{B200} = 3879.61$ lbs

Uniform Load = 50 plf

$$V_{B50} := \frac{M_{B50}}{\frac{4}{6}\cdot(h_B - t_B)} + q_{50}\cdot\frac{L_1}{12}$$

$V_{B50} = 3879.61$ lbs

Uniform Distributed Wind Load

$$V_{Bwind} := \frac{M_{Bwind}}{\frac{4}{6}\cdot(h_B - t_B)} + \frac{q_{wind}}{144}\cdot L_1\cdot(h + h_B)$$

$V_{Bwind} = 1697.12$ lbs

$V_{Bmax} := \max(V_{B200}, V_{B50}, V_{Bwind})$

$V_{Bmax} = 3879.61$ lbs

Check Resisting Couple in the Wall`s Molding:

$$F_B := \frac{M_{Bmax}}{h_B}$$

$F_B = 2029.27$ lbs

Geometric Inertia & Area in Wall`s Molding:

$$S_B := \frac{L_1\cdot t_B^2}{6}$$

$S_B = 5.78$ in^3

$A_B := L_1\cdot t_B$

$A_B = 40.80$ in^2

Check Bending Stress in Molding Wall:

$$f_b := \frac{M_{Bmax}}{S_B}$$

$f_b = 1727.34$ psi

BENDING := | "N.G."
"OK" if $f_b \le F_{b.}$

BENDING = "OK"

Check Shear Stress in Molding Wall:

$$f_v := \frac{F_B}{A_B}$$

$f_v = 49.74$ psi

SHEAR := | "N.G."
"OK" if $f_v \le F_{v.}$

SHEAR = "OK"

Check Combined Bending/Shear in Wall`s Monting:

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

$C_{BV} = 0.18$

COMBINED := | "N.G."
"OK" if $C_{BV} \le 1.00$

COMBINED = "OK"

ANCHOR AT BALCONY

Table 2 - Hilti KWIK HUS-EZ and KWIK HUS-EZ P design Strength with concrete / pullout failure in uncracked concrete[1,2,3,4]

Nominal anchor diameter in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
1/4 (6.4)	1-5/8 (41)	585 (2.6)	620 (2.8)	675 (3.0)	765 (3.4)	1,075 (4.8)	1,180 (5.2)	1,360 (6.0)	1,670 (7.4)
	2-1/2 (64)	1,525 (6.8)	1,670 (7.4)	1,930 (8.6)	2,365 (10.5)	2,235 (9.9)	2,450 (10.9)	2,825 (12.6)	3,460 (15.4)
3/8 (9.5)	1-5/8 (41)	910 (4.0)	1,000 (4.4)	1,155 (5.1)	1,415 (6.3)	980 (4.4)	1,075 (4.8)	1,245 (5.5)	1,520 (6.8)
	2-1/8 (54)	1,490 (6.6)	1,635 (7.3)	1,885 (8.4)	2,310 (10.3)	1,605 (7.1)	1,760 (7.8)	2,030 (9.0)	2,485 (11.1)
	2-1/2 (64)	1,980 (8.8)	2,165 (9.6)	2,505 (11.1)	3,065 (13.6)	2,130 (9.5)	2,335 (10.4)	2,695 (12.0)	3,300 (14.7)
	3-1/4 (83)	3,085 (13.7)	3,375 (15.0)	3,900 (17.3)	4,775 (21.2)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
1/2 (12.7)	2-1/4 (57)	1,645 (7.3)	1,800 (8.0)	2,080 (9.3)	2,550 (11.3)	1,770 (7.9)	1,940 (8.6)	2,240 (10.0)	2,745 (12.2)
	3 (76)	2,785 (12.4)	3,050 (13.6)	3,525 (15.7)	4,315 (19.2)	3,000 (13.3)	3,285 (14.6)	3,795 (16.9)	4,645 (20.7)
	4-1/4 (108)	5,070 (22.6)	5,555 (24.7)	6,415 (28.5)	7,855 (34.9)	10,920 (48.6)	11,965 (53.2)	13,815 (61.5)	16,920 (75.3)
5/8 (15.9)	3-1/4 (83)	3,240 (14.4)	3,550 (15.8)	4,100 (18.2)	5,025 (22.4)	3,490 (15.5)	3,825 (17.0)	4,415 (19.6)	5,410 (24.1)
	5 (127)	6,705 (29.8)	7,345 (32.7)	8,485 (37.7)	10,390 (46.2)	14,445 (64.3)	15,825 (70.4)	18,270 (81.3)	22,380 (99.6)
3/4 (19.1)	4 (102)	4,380 (19.5)	4,795 (21.3)	5,540 (24.6)	6,785 (30.2)	9,430 (41.9)	10,330 (45.9)	11,930 (53.1)	14,610 (65.0)
	6-1/4 (159)	9,345 (41.6)	10,235 (45.5)	11,820 (52.6)	14,475 (64.4)	20,125 (89.5)	22,045 (98.1)	25,455 (113.2)	31,175 (138.7)

3.3.6

Table 3 - Hilti KWIK HUS-EZ and KWIK HUS-EZ P design Strength with concrete / pullout failure in cracked concrete[1,2,3,4,5]

Nominal anchor diameter in. (mm)	Nominal embed. in. (mm)	Tension - ϕN_n				Shear - ϕV_n			
		f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.4 MPa) lb (kN)
1/4 (6.4)	1-5/8 (41)	300 (1.3)	315 (1.4)	345 (1.5)	390 (1.7)	765 (3.4)	835 (3.7)	965 (4.3)	1,180 (5.2)
	2-1/2 (64)	760 (3.4)	830 (3.7)	960 (4.3)	1,175 (5.2)	1,585 (7.1)	1,735 (7.7)	2,000 (8.9)	2,450 (10.9)
3/8 (9.5)	1-5/8 (41)	475 (2.1)	520 (2.3)	600 (2.7)	730 (3.2)	695 (3.1)	760 (3.4)	880 (3.9)	1,080 (4.8)
	2-1/8 (54)	1,055 (4.7)	1,155 (5.1)	1,335 (5.9)	1,635 (7.3)	1,135 (5.0)	1,245 (5.5)	1,440 (6.4)	1,760 (7.8)
	2-1/2 (64)	1,400 (6.2)	1,535 (6.8)	1,775 (7.9)	2,170 (9.7)	1,510 (6.7)	1,655 (7.4)	1,910 (8.5)	2,340 (10.4)
	3-1/4 (83)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2 (12.7)	2-1/4 (57)	1,035 (4.6)	1,135 (5.0)	1,310 (5.8)	1,605 (7.1)	1,115 (5.0)	1,220 (5.4)	1,410 (6.3)	1,725 (7.7)
	3 (76)	1,755 (7.8)	1,920 (8.5)	2,220 (9.9)	2,715 (12.1)	1,890 (8.4)	2,070 (9.2)	2,390 (10.6)	2,925 (13.0)
	4-1/4 (108)	3,190 (14.2)	3,495 (15.5)	4,040 (18.0)	4,945 (22.0)	6,875 (30.6)	7,530 (33.5)	8,695 (38.7)	10,650 (47.4)
5/8 (15.9)	3-1/4 (83)	2,040 (9.1)	2,235 (9.9)	2,580 (11.5)	3,165 (14.1)	2,200 (9.8)	2,410 (10.7)	2,780 (12.4)	3,405 (15.1)
	5 (127)	4,225 (18.8)	4,625 (20.6)	5,340 (23.8)	6,540 (29.1)	9,095 (40.5)	9,965 (44.3)	11,505 (51.2)	14,090 (62.7)
3/4 (19.1)	4 (102)	2,755 (12.3)	3,020 (13.4)	3,485 (15.5)	4,270 (19.0)	5,940 (26.4)	6,505 (28.9)	7,510 (33.4)	9,200 (40.9)
	6-1/4 (159)	5,885 (26.2)	6,445 (28.7)	7,440 (33.1)	9,115 (40.5)	12,670 (56.4)	13,880 (61.7)	16,030 (71.3)	19,630 (87.3)

1 1 See Section 3.1.9 to convert design strength value to ASD value.
2 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.
3 Apply spacing, edge distance, and concrete thickness factors in Tables 6 through 15 as necessary. Compare to the steel values in Table 4. The lesser of the values is to be used for the design.
4 Tabular values are for normal weight concrete only. For lightweight concrete multiply design strength by λa as follows: For sand-lightweight, λa = 0.68. For all-lightweight, λa = 0.60.
5 Tabular values are for static loads only. For seismic tension loads, multiply cracked concrete tabular values in tension by the following reduction factors:
1/4-in diameter by 1-5/8-in nominal embedment depth - $a_{N,seis}$ = 0.60
All other sizes - $a_{N,seis}$ = 0.75
No reduction needed for seismic shear. See Section 3.1.9 for additional information on seismic applications.

Table 8 - Load Adjustment Factors for 3/8-in. Diameter KWIK HUS-EZ and KWIK HUS-EZ P in Uncracked Concrete[1,4]

3/8-in. KH-EZ uncracked concrete	Spacing factor in tension f_{AN}				Edge distance factor in tension f_{RN}				Spacing factor in shear[3] f_{AV}				Edge distance in shear: ⊥ toward edge f_{RV}				Edge distance in shear: II to and away from edge f_{RV}				Conc. thickness factor in shear[4] f_{HV}			
Embedment h_{nom} in. (mm)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)
Spacing (s)/edge distance (c_a)/concrete thickness (h) - in. (mm): 1-1/2 (38)	n/a	n/a	n/a	n/a	0.58	0.62	0.63	0.57	n/a	n/a	n/a	n/a	0.49	0.32	0.25	0.08	0.58	0.62	0.50	0.17	n/a	n/a	n/a	n/a
2 (51)	n/a	n/a	n/a	n/a	0.76	0.75	0.75	0.66	n/a	n/a	n/a	n/a	0.75	0.49	0.38	0.13	0.76	0.75	0.75	0.26	n/a	n/a	n/a	n/a
2-1/4 (57)	0.84	0.74	0.70	0.65	0.86	0.82	0.81	0.70	0.65	0.62	0.60	0.55	0.90	0.59	0.46	0.16	0.90	0.82	0.81	0.31	n/a	n/a	n/a	n/a
2-1/2 (64)	0.88	0.77	0.72	0.67	0.95	0.91	0.88	0.75	0.67	0.63	0.61	0.55	1.00	0.69	0.54	0.18	1.00	0.91	0.88	0.37	n/a	n/a	n/a	n/a
3 (76)	0.95	0.82	0.77	0.70	1.00	1.00	1.00	0.85	0.71	0.66	0.63	0.56		0.90	0.71	0.24		1.00	1.00	0.48	n/a	n/a	n/a	n/a
3-1/4 (83)	0.99	0.85	0.79	0.72				0.90	0.73	0.67	0.64	0.57		1.00	0.80	0.27				0.54	0.95	n/a	n/a	n/a
3-1/2 (89)	1.00	0.88	0.81	0.73				0.95	0.74	0.68	0.65	0.58			0.89	0.30				0.61	0.98	n/a	n/a	n/a
4 (102)		0.93	0.86	0.77				1.00	0.78	0.71	0.68	0.59			1.00	0.37				0.74	1.00	0.91	0.84	n/a
4-1/2 (114)		0.99	0.90	0.80					0.81	0.73	0.70	0.60				0.44				0.88			0.89	n/a
4-3/4 (121)		1.00	0.93	0.82					0.83	0.75	0.71	0.60				0.48				0.96			0.91	0.639
5 (127)			0.95	0.83					0.84	0.76	0.72	0.61				0.52				1.00			0.94	0.655
6 (152)			1.00	0.90					0.91	0.81	0.76	0.63				0.68							1.00	0.718
7 (178)				0.97					0.98	0.86	0.81	0.65				0.86								0.775
8 (203)				1.00					1.00	0.91	0.85	0.67				1.00								0.829
9 (229)										0.97	0.90	0.69												0.879
10 (254)										1.00	0.94	0.71												0.927
11 (279)											0.98	0.74												0.972
12 (305)											1.00	0.76												1.000
14 (356)																								
16 (406)																								
18 (457)																								
20 (508)																								
24 (610)																								

8" spacing 3" edge distance
T= 3900 x 1.00 x 0.85 = 3315 lbs
V= 8400 x 0.67 x 0.24 = 1350.72 lbs
.
Ta= 3315 lbs/1.44 = 2302.08 lbs
Va= 1350.72 lbs/1.44 = 938 lbs
.
Okay for concrete connection at balcony

Table 9 - Load Adjustme… …d Concrete[1,4]

3/8-in. KH-EZ cracked concrete	Spacing… in tens… f_{AN}																…d away …edge f_{RV}				Conc. thickness factor in shear[4] f_{HV}			
Embedment h_{nom} in. (mm)	1-5/8 (41)	2-1/8 (54)																	2-1/2 (64)	3-1/4 (83)	1-5/8 (41)	2-1/8 (54)	2-1/2 (64)	3-1/4 (83)
Spacing (s)/edge distance (c_a)/concrete thickness (h) - in. (mm): 1-1/2 (38)	n/a	n/a	n/a	n/a	0.92	0.74	0.66	0.57	n/a	n/a	n/a	n/a	0.49	0.32	0.25	0.09	0.92	0.64	0.50	0.17	n/a	n/a	n/a	n/a
2 (51)	n/a	n/a	n/a	n/a	1.00	0.90	0.79	0.66	n/a	n/a	n/a	n/a	0.76	0.50	0.39	0.13	1.00	0.90	0.77	0.26	n/a	n/a	n/a	n/a
2-1/4 (57)	0.84	0.74	0.70	0.65	1.00	0.98	0.85	0.70	0.66	0.62	0.60	0.55	0.90	0.59	0.46	0.16	1.00	0.98	0.85	0.31	n/a	n/a	n/a	n/a
2-1/2 (64)	0.88	0.77	0.72	0.67	1.00	1.00	0.92	0.75	0.67	0.63	0.61	0.55	1.00	0.69	0.54	0.18	1.00	1.00	0.92	0.37	n/a	n/a	n/a	n/a
3 (76)	0.95	0.82	0.77	0.70	1.00		1.00	0.85	0.71	0.66	0.63	0.56	1.00	0.91	0.71	0.24	1.00	1.00	1.00	0.48	n/a	n/a	n/a	n/a
3-1/4 (83)	0.99	0.85	0.79	0.72				0.90	0.73	0.67	0.64	0.57		1.00	0.80	0.27				0.55	0.95	n/a	n/a	n/a
3-1/2 (89)	1.00	0.88	0.81	0.73				0.95	0.74	0.68	0.65	0.58			0.90	0.31				0.61	0.98	n/a	n/a	n/a
4 (102)		0.93	0.86	0.77				1.00	0.78	0.71	0.68	0.59			1.00	0.37				0.75	1.00	0.91	0.84	n/a
4-1/2 (114)		0.99	0.90	0.80					0.81	0.73	0.70	0.60				0.44				0.89		0.97	0.89	n/a
4-3/4 (121)		1.00	0.93	0.82					0.83	0.75	0.71	0.60				0.48				0.97		1.00	0.92	0.64
5 (127)			0.95	0.83					0.85	0.76	0.72	0.61				0.52				1.00			0.94	0.66
6 (152)			1.00	0.90					0.92	0.81	0.77	0.63				0.69							1.00	0.72
7 (178)				0.97					0.98	0.87	0.81	0.65				0.86								0.78
8 (203)				1.00					1.00	0.92	0.85	0.67				1.00								0.83
9 (229)										0.97	0.90	0.69												0.88
10 (254)										1.00	0.94	0.72												0.93
11 (279)											0.99	0.74												0.97
12 (305)											1.00	0.76												1.00
14 (356)												0.80												
16 (406)												0.85												
18 (457)												0.89												
20 (508)												0.93												
24 (610)												1.00												

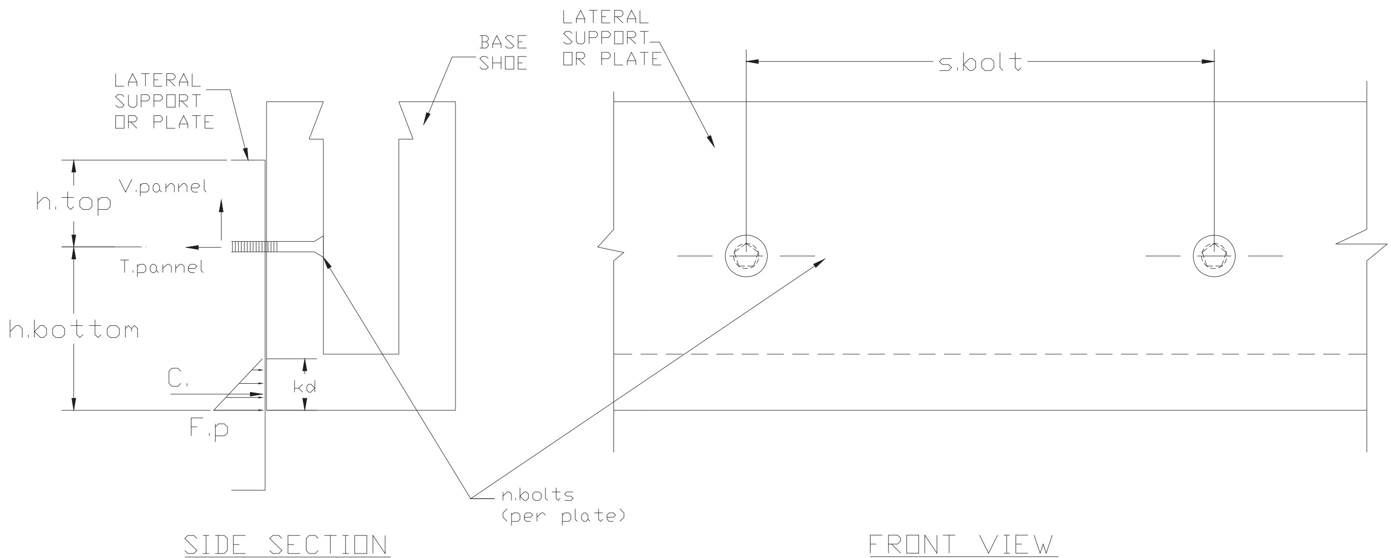
1 Linear interpolation not permitted.

2 When combining multiple load adjustment factors (e.g. for a 4 anchor pattern in a corner with thin concrete member) the design can become very conservative. To optimize the design, use Hilti PROFIS Anchor Design software or perform anchor calculation using design equations from ACI 318-14 Chapter 17.

3 Spacing factor reduction in shear, f_{AV}, assumes an influence of a nearby edge. If no edge exists, then $f_{AV} = f_{AN}$.

4 Concrete thickness reduction factor in shear, f_{HV}, assumes an influence of a nearby edge. If no edge exists, then f_{HV} = 1.0.

▭ If a reduction factor value is in a shaded cell, this indicates that this specific edge distance may not be permitted with a certain spacing (or vice versa). Check table 5 and figure 2 of this section to calculate permissable edge distance, spacing and concrete thickness combinations.



Check Anchors:

$s_{bolt} := 8.00$ Spacing of Bolts Per Pannels

$h_{top} := 2.40$ Height of Support at Top (in)

$h_{bottom} := 2.4$ Height of Support at Bottom (in)

$T_{bolt} := 2320.0$ Allowable Tension Per Bolt (lbs)

$V_{bolt} := 938.0$ Allowable Shear Per Bolt (lbs)

$F_p := 5000.00$ Allowable Compressive Strength in Support (psi)

Minimum Lateral Support Heigth (h):

$h_{min} := \min(h_{top}, h_{bottom})$ $h_{min} = 2.40$ in

Length of Compression Zone:

$$kd := \left[\frac{F_p \cdot L_1 \cdot (h_{min})}{2} - \sqrt{\frac{(F_p \cdot L_1)^2 \cdot h_{min}^2}{4} - \frac{4 \cdot (M_{Bmax}) \cdot F_p \cdot L_1}{6}}\right] \cdot \frac{3}{F_p \cdot L_1}$$

$kd = 0.03$ in

Tensile Load on Cap Screws Per Width of Glass Pannel:

$T_{pannel} := 0.5 \cdot F_p \cdot kd \cdot L_1$ $T_{pannel} = 4180.22$ lbs

Shear Load on Cap Screws Per Width of Glass Pannel:

$V_{pannel} := V_{max}$ $V_{pannel} = 200.00$ lbs

Check Compressive Stress in Support:

$f_p := \text{floor}\left(\frac{2T_{pannel}}{kd \cdot L_1}\right)$ $f_p = 5000.00$ psi

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

Number of Anchors Per Loaded Triangle:

$N_{anchors} := \text{floor}\left(\frac{L_1}{s_{bolt}}\right)$ $N_{anchors} = 6.00$ bolts

Check Tension in Anchors:

$T_d := \frac{T_{pannel}}{N_{anchors}}$ $T_d = 696.70$ lbs

$TENSION := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } T_{bolt} \geq T_d \end{vmatrix}$ TENSION = "OK"

Check Shear in Anchors:

$V_d := \frac{V_{pannel}}{N_{anchors}}$ $V_d = 33.33$ lbs

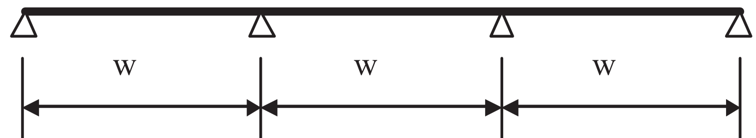
$SHEAR := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } V_{bolt} \geq V_d \end{vmatrix}$ SHEAR = "OK"

Check Combined Tension/Shear in Anchors:

$C_{TV} := \left(\frac{T_d}{T_{bolt}}\right) + \left(\frac{V_d}{V_{bolt}}\right)$ $C_{TV} = 0.34$

$COMBINED_1 := \begin{vmatrix} \text{"N.G."} \\ \text{"OK"} \quad \text{if } C_{TV} \leq 1.00 \end{vmatrix}$ $COMBINED_1$ = "OK"

Top Railing Design:



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

Handrail Material Data:

$F_{b.top} := 1700$ Top Railing Allowable Bending Stress (psi)

$F_{v.top} := 175$ Top Railing Allowable Shear Stress (psi)

$Sx_{top} := 0.48$ Top Railing Inertia Modulus for Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.53$ Top Railing Inertia Modulus for Horizontal Loads $\left(in^3\right)$

$A_{top} := 1.75$ Top Railing Area $\left(in^2\right)$

Actual Moment:

Concentrated Load = 200 lbs.

$$M_{200.top} := \frac{P_{200} \cdot 0.5}{5}$$

$M_{200.top} = 20.00$ lb − in

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

Uniform Load = 50 plf

$$M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot 0.5^2$$

$M_{50.top} = 0.11$ lb − in

$M_{max.top} = 20.00$ lb − in

Actual Shear:

Concentrated Load = 200 lbs.

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

$V_{200.top} = 200.00$ lbs

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

Uniform Load = 50 plf

$$V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot 0.5$$

$V_{50.top} = 1.25$ lbs

$V_{max.top} = 200.00$ lbs

Section Required:

Bending Design:
Section Modulus Required

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

$Shr = 0.01$ in^3

Shear Design:
Area Required

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

$Ahr = 1.71$ in^2

Section Provided:

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

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