CALCULATIONS
FOR
STRUCTURAL ANALYSIS
Of

HPER Building Bike Canopy

Logan, UT

8/1/2023

PROFESSIONAL STRUCTURAL ENGINEER
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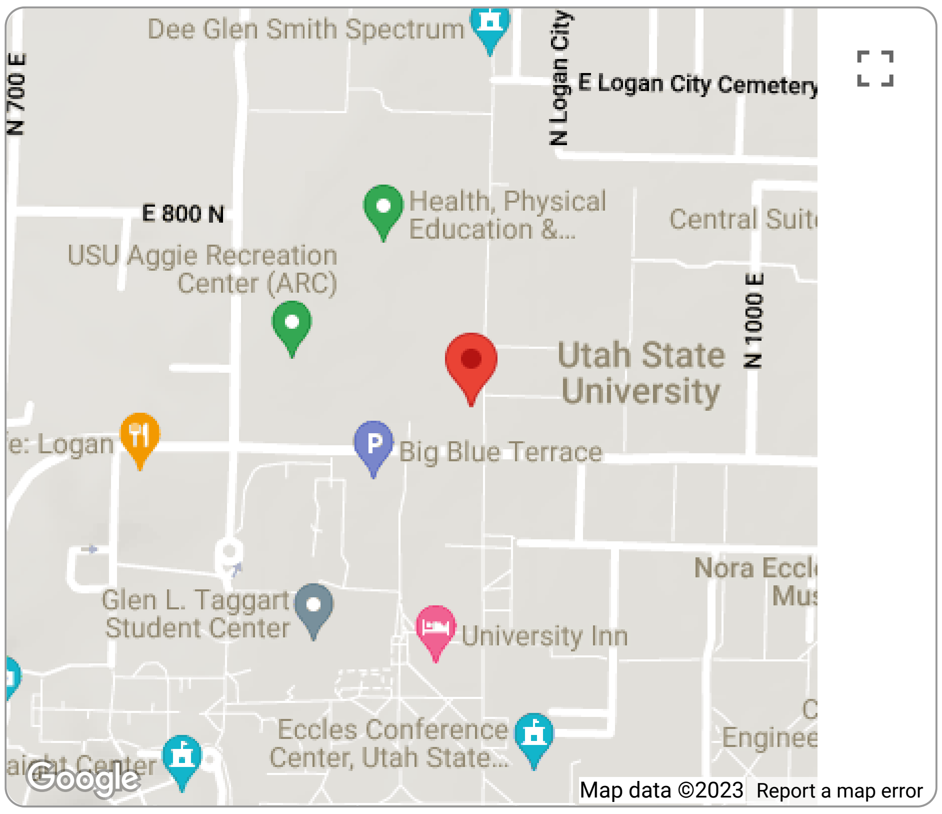
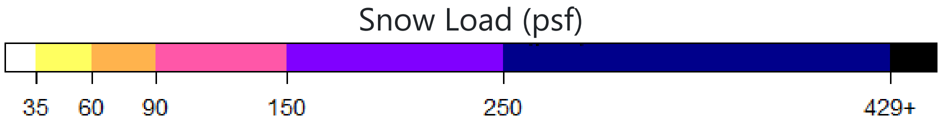
HYPR Building



Latitude: 41.745
Longitude: -111.812
Elevation: 4,726 ft

Ground Snow Load:
47 psf / 2.26 kPa



*This document is not legally binding. The user is urged to verify ground snow load values with the local authority having jurisdiction.

These ground snow load values represent 50-year ground snow load estimated value at a 2% probability of exceedance for the location given. The grid used in the map is 3350ft by 3350ft. Elevations for these grid cells were estimated by aggregating data from 100ft by 100ft USGS digital elevation models and may not coincide with the actual site elevation. These predictions are calculated using the process outlined in The Utah Snow Load Study.[1]

Final predictions given are bounded at a lower limit for a minimum ground snow load of 21 psf to meet ASCE 7. Estimated values for snow loads at elevations significantly higher than all nearby stations lead to unreasonably high snow load estimates, therefore, the predictions in the map are not allowed to extend beyond the highest 50-year station ground snow load of 429 psf. Elevations over 9,000 ft are also considered less accurate due to the limited number of stations at these elevations. The results shown in this report have included a warning if the results have reached or exceeded the upper limit.

While great efforts have been made to ensure these predictions are as accurate as possible, designers must use expert judgement to ensure that such predictions are appropriate for their particular project. The SEAU and the authors cannot accept responsibility for prediction errors or any consequences resulting therefrom.

1 Bean, Brennan; Maguire, Marc; and Sun, Yan, "The Utah Snow Load Study" (2018). Civil and Environmental Engineering Faculty Publications. Paper 3589.

ATC Hazards by Location

Search Information

Coordinates:	41.74483650096447, -111.81157280926207
Elevation:	4784 ft
Timestamp:	2023-07-31T15:37:24.009Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.038	MCE_R ground motion (period=0.2s)
S_1	0.346	MCE_R ground motion (period=1.0s)
S_{MS}	1.245	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.83	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.898	Coefficient of risk (0.2s)
CR_1	0.908	Coefficient of risk (1.0s)
PGA	0.448	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.538	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.038	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.155	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.958	Factored deterministic acceleration value (0.2s)
S1RT	0.346	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.381	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.303	Factored deterministic acceleration value (1.0s)
PGAd	1.192	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Coordinates:	41.74483650096447, -111.81157280926207
Elevation:	4784 ft
Timestamp:	2023-07-31T15:35:50.375Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	74 mph
MRI 25-Year	80 mph
MRI 50-Year	85 mph
MRI 100-Year	89 mph
Risk Category I	98 mph
Risk Category II	103 mph
Risk Category III	110 mph
Risk Category IV	114 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Owner/Project: USU Bike Rack HYPR | By: MJM | Date: 7/31/23 | Project No.: 14-123-0088
Feature USU Bike Rack HYPR | Chk'd By: | Date: | Sheet:

Wind Design Data - ASCE 7-16

Code =	2021 IBC	
Risk Category =	II	IBC Table 1604.5
V_{ult} (mph)	103	Basic Wind Speed, IBC Figure 1609.3
V_{asd} (mph)	80	IBC Eqn. 16-33
K_d =	0.85	ASCE Table 26.6-1
Surface Roughness	C	ASCE 7, Sec. 26.7.2
Exposure	C	ASCE 7, Sec. 26.7.3
K_{zt} =	1.000	ASCE 7, Figure 26.8-1
G =	0.85	ASCE 7, Sec. 26.11
Enclosure =	Open Building	ASCE 7, Sec. 26.12
GC_{pi} =	0.00	(+/-) ASCE 7, Table 26.13-1
K_e =	0.86	ASCE 7, Table 26.9-1
K_z =	0.85	ASCE 7, Table 26.10-1
q_z (psf)=	16.9	

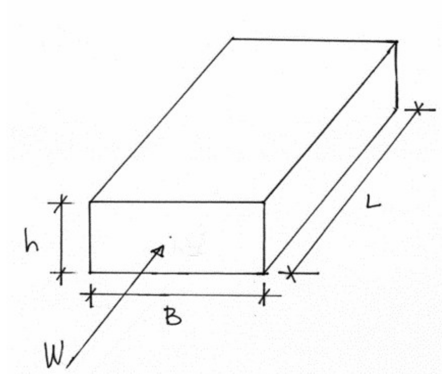
Roof Slope =	2	in/ft
Slope Angle =	9.5	degrees

ASCE 7, Chapter 27, Part 1, Main Wind Force Resisting System

Wind Direction 1: N-S (Wind Direction 1 is normal to the high and low edge of roof)
Wind Direction 2: E-W (Wind Direction 2 is parallel to the high and low edge of the roof)

C_p , Wind on Walls

Direction 1		Direction 2	
L (ft) =	10	L (ft) =	26
B (ft) =	26	B (ft) =	10
Dir. 1: L/B =	0.4	Dir. 2: L/B =	2.6



*Wind Direction 1 is required to be defined as normal to the high and low edge of the roof for this calculation.

FORSGREN Associates Inc.

Owner/Project:	USU Bike Rack HYPR	By:	MJM	Date:	7/31/23	Project No.:	14-23-0088
Feature	Roof Loads	Chk'd By:		Date:		Sheet:	

Monoslope Free Roof, ASCE Chapter 27, Part 1, MWFRS

Description: Merrill Library Bike Canopy

L =	10.00	ft	Length, Perpendicular to Wind Direction 1
H_P =	1.67	ft	Vertical Monoslope Dimension
L =	10.58	ft	Horizontal Monoslope Dimension

Roof

Rise =	2	in/ft	Rise of roof
W =	10.7	ft	Width from eave to peak along slope
L_R =	10.00	ft	Length of Roof
A =	107.26	ft^2	Area of the roof normal to roof surface

Roof Pressures Leeward and Windward

	q	G	C_{NW}	C_{NL}	P_{NW}	P_{NL}	
0 Degrees, Case A	19.62	0.85	-0.60	-1.00	-10.01	-16.68	$p = q*G*C_N$
0 Degrees, Case B	19.62	0.85	-1.40	0.00	-23.35	0.00	$p = q*G*C_N$
180 Degrees, Case A	19.62	0.85	0.90	1.50	15.01	25.02	$p = q*G*C_N$
180 Degrees, Case B	19.62	0.85	1.60	0.30	26.69	5.00	$p = q*G*C_N$

Roof Horizontal Pressures Component

	P_{NW}	P_{NL}		P_{NW} * A /2	P_{LW} * A /2		
0 Degrees, Case A	-1.65	-2.74	psf (p*sin("roof angle"))	-88	-147	lbs	-235
0 Degrees, Case B	-3.84	0.00	psf (p*sin("roof angle"))	-206	0	lbs	-206
180 Degrees, Case A	2.47	4.11	psf (p*sin("roof angle"))	132	221	lbs	353
180 Degrees, Case B	4.39	0.82	psf (p*sin("roof angle"))	235	44	lbs	279

Roof Vertical Pressures Component

	P_{NW}	P_{NL}		P_{NW} * A /2	P_{LW} * A /2		
0 Degrees, Case A	-9.87	-16.45	psf (p*cos("roof angle"))	-529	-882	lbs	-1412
0 Degrees, Case B	-23.03	0.00	psf (p*cos("roof angle"))	-1235	0	lbs	-1235
180 Degrees, Case A	14.81	24.68	psf (p*cos("roof angle"))	794	1323	lbs	2118
180 Degrees, Case B	26.32	4.94	psf (p*cos("roof angle"))	1412	265	lbs	1676

0 Degree, Case A, Roof F_H =	-235	lbs	0 Degree, Case A, Roof F_V =	-1412	lbs
0 Degree, Case B, Roof F_H =	-206	lbs	0 Degree, Case B, Roof F_V =	-1235	lbs
180 Degree, Case A, Roof F_H =	353	lbs	180 Degree, Case A, Roof F_V =	2118	lbs
180 Degree, Case B, Roof F_H =	279	lbs	180 Degree, Case B, Roof F_V =	1676	lbs
Maximum, Roof F_H =	353	lbs	Maximum, Roof F_V =	2118	lbs

Job:USU HPER Bike Rack

Engineer:MJM

Job#:14-23-0088

Date:7/31/2023

Dead Load Calculation

$Insulation := 0\ psf$

$Membrain := 0\ psf$

$Deck := 3\ psf$

HSS4X2X1/4

Joists/Purlins

$J_{spacing} := 2\ ft$

$J_{wt} := 9.66\ plf$

$$Joists := \frac{J_{wt}}{J_{spacing}} = 4.83\ psf$$

HSS6X4X1/4

Girders

$G_{spacing} := 12\ ft$

$G_{wt} := 15.62\ plf$

$$Girders := \frac{G_{wt}}{G_{spacing}} = 1.302\ psf$$

Dead load Calc.

$$D := Insulation + Membrain + Deck + Joists + Girders = 9.13\ psf$$

USE DEAD LOAD OF 10 PSF

Snow Load

$Pg := 47\ psf$ $C_t := 1.2$ $C_e := 1.0$ $C_s := 1.0$ $I_s := 1.00$

Flat roof $P_f := 0.7 \cdot C_t \cdot C_e \cdot Pg \cdot I_s = 39.48\ psf$

Sloped roof $P_s := P_f \cdot C_s = 39.48\ psf$

Seismic

$S_s := 1.068$

$S_1 := 0.346$

$I_e := 1.0$

$F_a := 1.2$

$F_v := NA$

$R := 1.25$

$S_{MS} := F_a \cdot S_s = 1.2816$

$S_{M1} := F_v \cdot F_1 = ∎$

$S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.8544$

$S_{D1} := \frac{2}{3} \cdot S_{M1} = ∎$

$$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.6835$$

$D := 10\ psf$

$Coll := 0\ psf$

$S := 40\ psf$

$Wall_{wt} := 0\ psf$

$Wall_h := 0\ ft$

Roof Dimensions

$Roof_w := 10\ ft$

$Roof_L := 25.83\ ft$

$IntWall_L := 0\ ft$

$\Omega := 1.25$ $\rho := 1.3$

SEISMIC Base Shear

$$V := C_s \cdot \left(\left(\left(Roof_w\right) \cdot \left(Roof_L\right)\right) \cdot \left(D + 0.6 \cdot Coll + 0.2 \cdot S\right)\right) = 3.178\ kip$$

$$Load_{shortE} := \frac{V}{Roof_w} = 0.3178\ \frac{kip}{ft} \qquad Load_{longE} := \frac{V}{Roof_L} = 0.123\ \frac{kip}{ft}$$

Seismic Load Longitudinal to Each Column $Load_{Col.Long} := 0.333 \cdot V \cdot \rho = 1.3757\ kip$

Seismic Lateral to Each Column $Load_{Col.Lat} := 0.42 \cdot V \cdot \rho = 1.7352\ kip$

$Height_{col} := 9.25\ ft$

Design Moment to columns: $M_{base} := Load_{Col.Lat} \cdot Height_{col} = 16.0503\ kip\ ft$

Dead Load to Center Column $D_{Col} := 10\ ft \cdot 10.58\ ft \cdot D = 1.058\ kip$

$Moment_D := 0.5\ ft \cdot D_{Col} = 0.529\ kip\ ft$

Snow Load to Center Column $D_{Col} := 10\ ft \cdot 10.58\ ft \cdot S = 4.232\ kip$

$Moment_D := 0.5\ ft \cdot D_{Col} = 2.116\ kip\ ft$

Moment to Footing/Anchors

$$Moment_{BCfooting} := M_{base} = 16.0503\ kip\ ft$$

24" Dia sono tube at 5' of depth

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

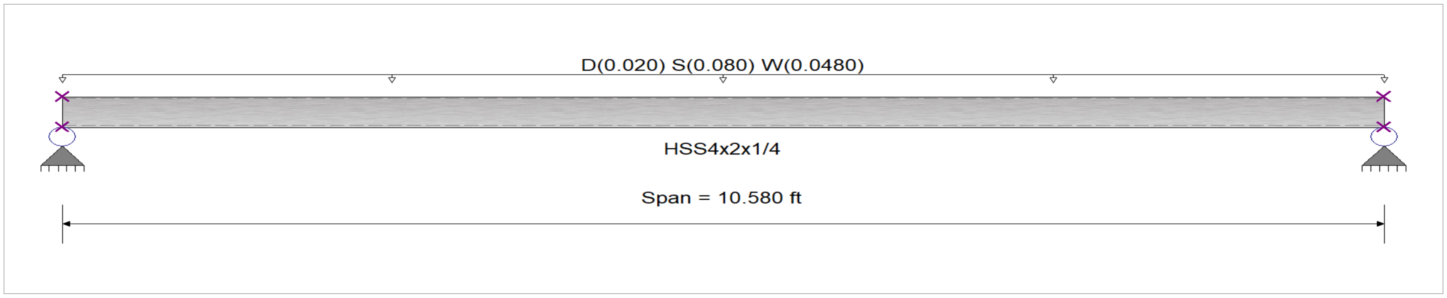
DESCRIPTION: Purlin General Span (Merrill)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.010, S = 0.040, W = 0.0240 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.211** : 1	Maximum Shear Stress Ratio =	**0.021** : 1
Section used for this span	**HSS4x2x1/4**	Section used for this span	**HSS4x2x1/4**
Ma : Applied	1.422 k-ft	Va : Applied	0.5375 k
Mn / Omega : Allowable	6.749 k-ft	Vn/Omega : Allowable	25.423 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.174 in	Ratio =	729	>=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<240	n/a
Max Downward Total Deflection	0.221 in	Ratio =	575	>=180	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 10.58 ft	1	0.041	0.004	0.28		0.28	11.27	6.75	1.14	1.00	0.11	42.46	25.42
+D+S													
Dsgn. L = 10.58 ft	1	0.207	0.021	1.40		1.40	11.27	6.75	1.14	1.00	0.53	42.46	25.42
+D+0.750S													
Dsgn. L = 10.58 ft	1	0.166	0.017	1.12		1.12	11.27	6.75	1.14	1.00	0.42	42.46	25.42
+D+0.60W													
Dsgn. L = 10.58 ft	1	0.101	0.010	0.68		0.68	11.27	6.75	1.14	1.00	0.26	42.46	25.42
+D+0.450W													
Dsgn. L = 10.58 ft	1	0.086	0.009	0.58		0.58	11.27	6.75	1.14	1.00	0.22	42.46	25.42
+D+0.750S+0.450W													
Dsgn. L = 10.58 ft	1	0.211	0.021	1.42		1.42	11.27	6.75	1.14	1.00	0.54	42.46	25.42
+0.60D+0.60W													
Dsgn. L = 10.58 ft	1	0.085	0.008	0.57		0.57	11.27	6.75	1.14	1.00	0.22	42.46	25.42
+0.60D													
Dsgn. L = 10.58 ft	1	0.025	0.002	0.17		0.17	11.27	6.75	1.14	1.00	0.06	42.46	25.42

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.2210	5.320		0.0000	0.000

Steel Beam

Project File: USU HYPR Bike Rack.ec6

DESCRIPTION: Purlin General Span (Merrill)

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.537	0.537
Max Upward from Load Combinations	0.537	0.537
Max Upward from Load Cases	0.423	0.423
D Only	0.106	0.106
+D+S	0.529	0.529
+D+0.750S	0.423	0.423
+D+0.60W	0.258	0.258
+D+0.450W	0.220	0.220
+D+0.750S+0.450W	0.537	0.537
+0.60D+0.60W	0.216	0.216
+0.60D	0.063	0.063
S Only	0.423	0.423
W Only	0.254	0.254

Pole Footing Embedded in Soil

Project File: USU HYPR Bike Rack.ec6

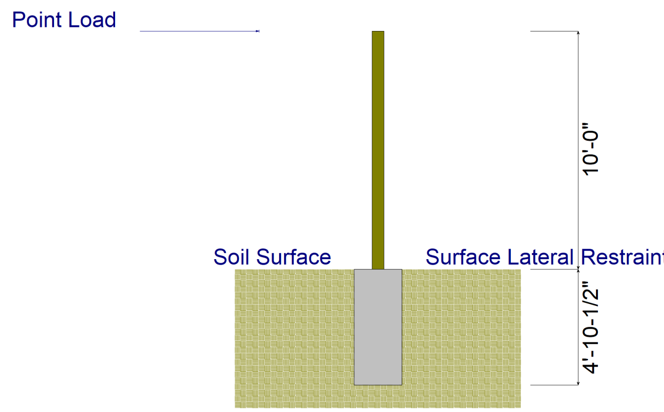
DESCRIPTION: Post (Strong Axis)

Code References

Calculations per IBC 2021 1807.3, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	24.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf



Controlling Values

Governing Load Combination	+D+0.70E
Lateral Load	1.219 k
Moment	12.723 k-ft

Restraint @ Ground Surface

Pressure at Depth

Actual	**1,137.63** psf
Allowable	**1,218.75** psf
Surface Retraint Force	6,765.32 lbs
Minimum Required Depth	**4.875 ft**
Footing Base Area	3.142 ft^2
Maximum Soil Pressure	404.845 ksf

Applied Loads

	Lateral Concentrated Load (k)	Lateral Distributed Loads (k	Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k	k/ft	0.5290 k-ft	1.058 k
Lr : Roof Live	k	k/ft	k-ft	k
L : Live	k	k/ft	k-ft	k
S : Snow	k	k/ft	2.116 k-ft	4.232 k
W : Wind	0.3530 k	k/ft	k-ft	2,118.0 k
E : Earthquake	1.742 k	k/ft	k-ft	k
H : Lateral Earth	k	k/ft	k-ft	k
Load distance above ground surface	10.0 ft	TOP of Load above ground surface ft		
		BOTTOM of Load above ground surface ft		

Load Combination Results

Load Combination	Forces @ Ground Surface Loads - (k)	Moments - (ft-k)	Required Depth - (ft)	Pressure at Depth Actual - (psf)	Allow - (psf)	Soil Increase Factor
D Only	0.000	0.529	1.75	367.1	437.5	1.000
+D+S	0.000	2.645	2.88	680.0	718.8	1.000
+D+0.750S	0.000	2.116	2.63	652.6	656.3	1.000
+D+0.60W	0.212	2.647	2.88	680.5	718.8	1.000
+D+0.450W	0.159	2.118	2.63	653.0	656.3	1.000
+D+0.750S+0.450W	0.159	3.705	3.25	745.3	812.5	1.000
+0.60D+0.60W	0.212	2.435	2.75	684.3	687.5	1.000
+D+0.70E	1.219	12.723	4.88	1,137.6	1,218.8	1.000
+D+0.750S+0.5250E	0.915	11.262	4.63	1,118.7	1,156.3	1.000

DESCRIPTION: Post (Strong Axis)

+0.60D+0.70E	1.219	12.511	4.75	1,178.4	1,187.5	1.000

Steel Beam

Project File: USU HYPR Bike Rack.ec6

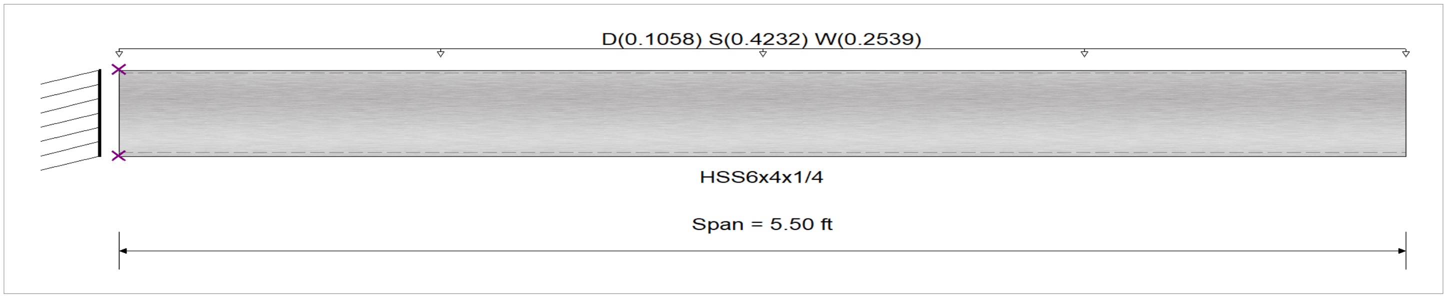
DESCRIPTION: Main Beam Uniform (Long Cantilever)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.010, S = 0.040, W = 0.0240 ksf, Tributary Width = 10.580 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.415** : 1	Maximum Shear Stress Ratio =	**0.072** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	8.129 k-ft	Va : Applied	2.956 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.138 in	Ratio =	958	>=180	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<180	n/a
Max Downward Total Deflection	0.175 in	Ratio =	755	>=120	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	0 in	Ratio =	0	<120	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 5.50 ft	1	0.082	0.014		-1.60	1.60	32.70	19.58	1.00	1.00	0.58	68.18	40.83
+D+S													
Dsgn. L = 5.50 ft	1	0.409	0.071		-8.00	8.00	32.70	19.58	1.00	1.00	2.91	68.18	40.83
+D+0.750S													
Dsgn. L = 5.50 ft	1	0.327	0.057		-6.40	6.40	32.70	19.58	1.00	1.00	2.33	68.18	40.83
+D+0.60W													
Dsgn. L = 5.50 ft	1	0.199	0.035		-3.90	3.90	32.70	19.58	1.00	1.00	1.42	68.18	40.83
+D+0.450W													
Dsgn. L = 5.50 ft	1	0.170	0.030		-3.33	3.33	32.70	19.58	1.00	1.00	1.21	68.18	40.83
+D+0.750S+0.450W													
Dsgn. L = 5.50 ft	1	0.415	0.072		-8.13	8.13	32.70	19.58	1.00	1.00	2.96	68.18	40.83
+0.60D+0.60W													
Dsgn. L = 5.50 ft	1	0.167	0.029		-3.26	3.26	32.70	19.58	1.00	1.00	1.19	68.18	40.83
+0.60D													
Dsgn. L = 5.50 ft	1	0.049	0.009		-0.96	0.96	32.70	19.58	1.00	1.00	0.35	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.1749	5.500		0.0000	0.000

Steel Beam

Project File: USU HYPR Bike Rack.ec6

DESCRIPTION: Main Beam Uniform (Long Cantilever)

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.956	
Max Upward from Load Combinations	2.956	
Max Upward from Load Cases	2.328	
D Only	0.582	
+D+S	2.910	
+D+0.750S	2.328	
+D+0.60W	1.420	
+D+0.450W	1.210	
+D+0.750S+0.450W	2.956	
+0.60D+0.60W	1.187	
+0.60D	0.349	
S Only	2.328	
W Only	1.397	

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

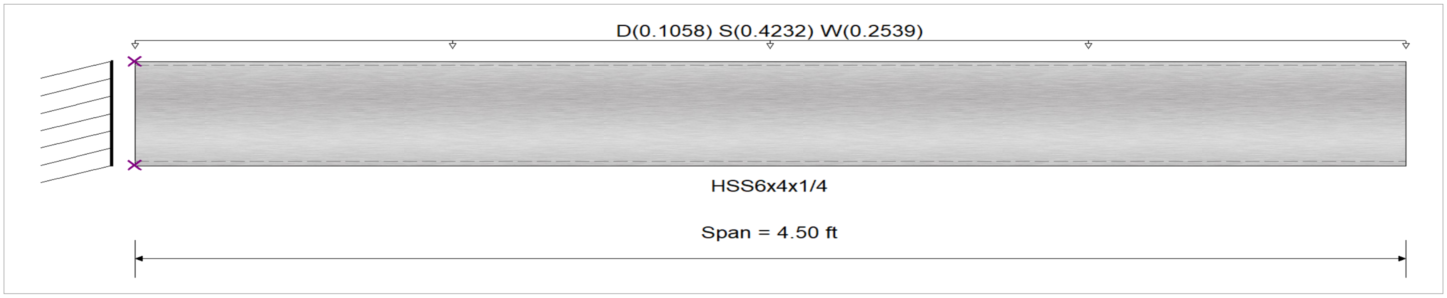
DESCRIPTION: Main Beam Uniform (Short Cantilever)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.010, S = 0.040, W = 0.0240 ksf, Tributary Width = 10.580 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.278** : 1	Maximum Shear Stress Ratio =	**0.059** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	5.442 k-ft	Va : Applied	2.419 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.062 in	Ratio =	1,749	>=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<240	n/a
Max Downward Total Deflection	0.078 in	Ratio =	1378	>=180	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 4.50 ft	1	0.055	0.012		-1.07	1.07	32.70	19.58	1.00	1.00	0.48	68.18	40.83
+D+S													
Dsgn. L = 4.50 ft	1	0.274	0.058		-5.36	5.36	32.70	19.58	1.00	1.00	2.38	68.18	40.83
+D+0.750S													
Dsgn. L = 4.50 ft	1	0.219	0.047		-4.28	4.28	32.70	19.58	1.00	1.00	1.90	68.18	40.83
+D+0.60W													
Dsgn. L = 4.50 ft	1	0.133	0.028		-2.61	2.61	32.70	19.58	1.00	1.00	1.16	68.18	40.83
+D+0.450W													
Dsgn. L = 4.50 ft	1	0.114	0.024		-2.23	2.23	32.70	19.58	1.00	1.00	0.99	68.18	40.83
+D+0.750S+0.450W													
Dsgn. L = 4.50 ft	1	0.278	0.059		-5.44	5.44	32.70	19.58	1.00	1.00	2.42	68.18	40.83
+0.60D+0.60W													
Dsgn. L = 4.50 ft	1	0.112	0.024		-2.19	2.19	32.70	19.58	1.00	1.00	0.97	68.18	40.83
+0.60D													
Dsgn. L = 4.50 ft	1	0.033	0.007		-0.64	0.64	32.70	19.58	1.00	1.00	0.29	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.0784	4.500		0.0000	0.000

Steel Beam

Project File: USU HYPR Bike Rack.ec6

DESCRIPTION: Main Beam Uniform (Short Cantilever)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.419	
Max Upward from Load Combinations	2.419	
Max Upward from Load Cases	1.904	
D Only	0.476	
+D+S	2.381	
+D+0.750S	1.904	
+D+0.60W	1.162	
+D+0.450W	0.990	
+D+0.750S+0.450W	2.419	
+0.60D+0.60W	0.971	
+0.60D	0.286	
S Only	1.904	
W Only	1.143	

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

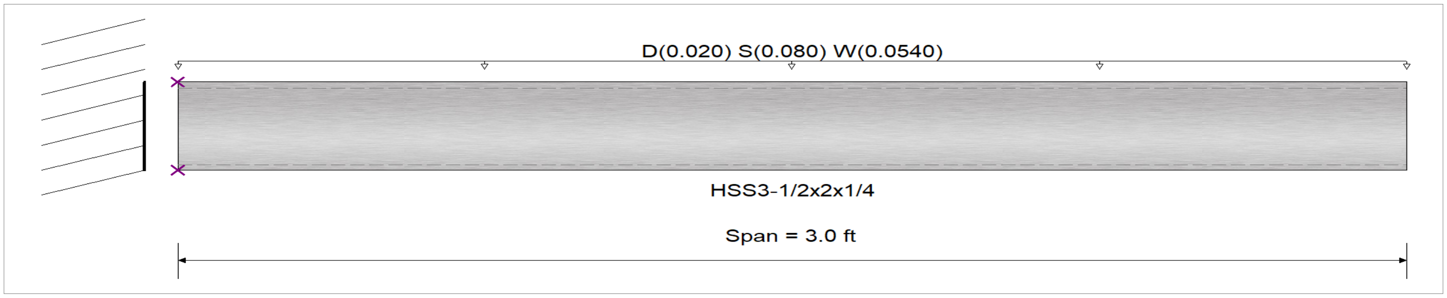
DESCRIPTION: Purlin Cantilever

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.010, S = 0.040, W = 0.0270 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.087** : 1	Maximum Shear Stress Ratio =	**0.015** : 1
Section used for this span	**HSS3-1/2x2x1/4**	Section used for this span	**HSS3-1/2x2x1/4**
Ma : Applied	0.469 k-ft	Va : Applied	0.3129 k
Mn / Omega : Allowable	5.417 k-ft	Vn/Omega : Allowable	21.572 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.015 in	Ratio = 4,737	>=180	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio = 0	<180	n/a
Max Downward Total Deflection	0.020 in	Ratio = 3634	>=120	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	0 in	Ratio = 0	<120	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 3.00 ft	1	0.017	0.003		-0.09	0.09	9.05	5.42	1.00	1.00	0.06	36.03	21.57
+D+S													
Dsgn. L = 3.00 ft	1	0.083	0.014		-0.45	0.45	9.05	5.42	1.00	1.00	0.30	36.03	21.57
+D+0.750S													
Dsgn. L = 3.00 ft	1	0.066	0.011		-0.36	0.36	9.05	5.42	1.00	1.00	0.24	36.03	21.57
+D+0.60W													
Dsgn. L = 3.00 ft	1	0.044	0.007		-0.24	0.24	9.05	5.42	1.00	1.00	0.16	36.03	21.57
+D+0.450W													
Dsgn. L = 3.00 ft	1	0.037	0.006		-0.20	0.20	9.05	5.42	1.00	1.00	0.13	36.03	21.57
+D+0.750S+0.450W													
Dsgn. L = 3.00 ft	1	0.087	0.015		-0.47	0.47	9.05	5.42	1.00	1.00	0.31	36.03	21.57
+0.60D+0.60W													
Dsgn. L = 3.00 ft	1	0.037	0.006		-0.20	0.20	9.05	5.42	1.00	1.00	0.13	36.03	21.57
+0.60D													
Dsgn. L = 3.00 ft	1	0.010	0.002		-0.05	0.05	9.05	5.42	1.00	1.00	0.04	36.03	21.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.0198	3.000		0.0000	0.000

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC.

DESCRIPTION: Purlin Cantilever

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.313	
Max Upward from Load Combinations	0.313	
Max Upward from Load Cases	0.240	
D Only	0.060	
+D+S	0.300	
+D+0.750S	0.240	
+D+0.60W	0.157	
+D+0.450W	0.133	
+D+0.750S+0.450W	0.313	
+0.60D+0.60W	0.133	
+0.60D	0.036	
S Only	0.240	
W Only	0.162	

Project Title: USU Bike Canopy HYPR
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

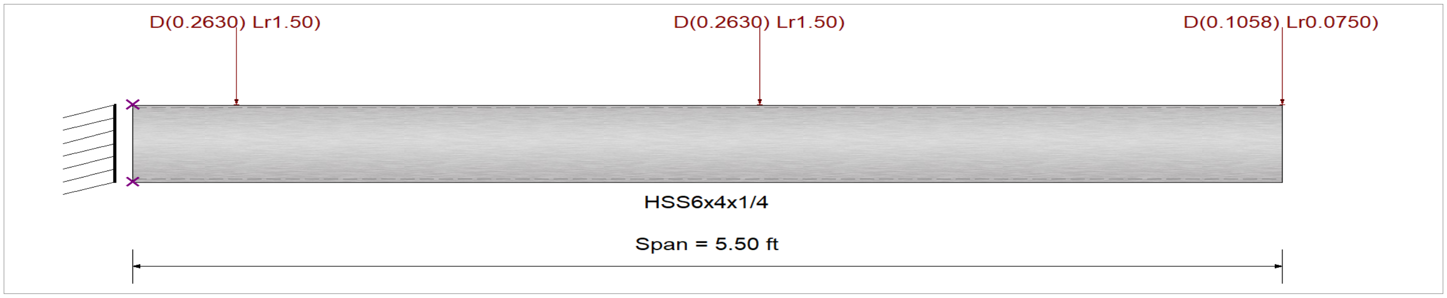
DESCRIPTION: Left Side (Long) Main Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 0.1058, Lr = 0.0750 k @ 5.50 ft

Point Load : D = 0.2630, Lr = 1.50 k @ 3.0 ft

Point Load : D = 0.2630, Lr = 1.50 k @ 0.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.366** : 1	Maximum Shear Stress Ratio =	**0.091** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	7.165 k-ft	Va : Applied	3.707 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.101 in	Ratio =	1,305	>=180	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<180	n/a
Max Downward Total Deflection	0.133 in	Ratio =	989	>=120	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<120	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 5.50 ft	1	0.077	0.015		-1.50	1.50	32.70	19.58	1.00	1.00	0.63	68.18	40.83
+D+Lr													
Dsgn. L = 5.50 ft	1	0.366	0.091		-7.16	7.16	32.70	19.58	1.00	1.00	3.71	68.18	40.83
+D+0.750Lr													
Dsgn. L = 5.50 ft	1	0.294	0.072		-5.75	5.75	32.70	19.58	1.00	1.00	2.94	68.18	40.83
+0.60D													
Dsgn. L = 5.50 ft	1	0.046	0.009		-0.90	0.90	32.70	19.58	1.00	1.00	0.38	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1335	5.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.707	

Steel Beam

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Left Side (Long) Main Beam

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinations	3.707	
Max Upward from Load Cases	3.075	
D Only	0.632	
+D+Lr	3.707	
+D+0.750Lr	2.938	
+0.60D	0.379	
Lr Only	3.075	

Steel Beam

Project File: USU Bike Racks.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC.

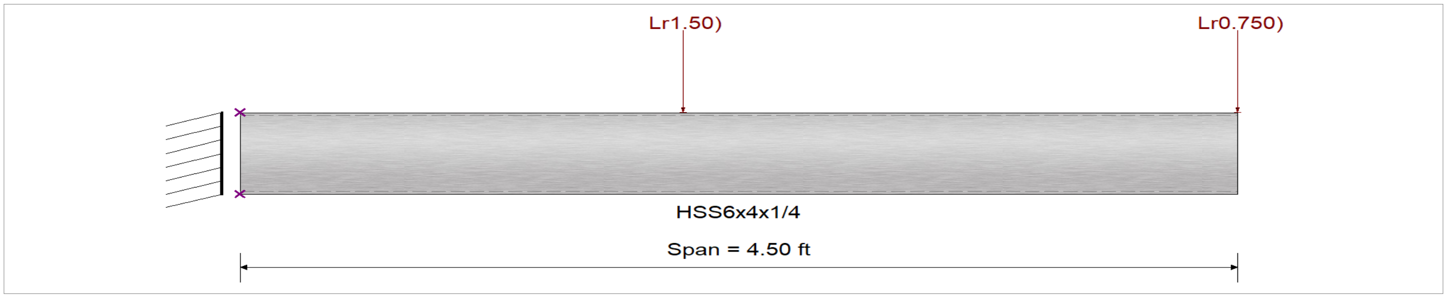
DESCRIPTION: Right Side (Short) Main Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : Lr = 0.750 k @ 4.50 ft

Point Load : Lr = 1.50 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.326** : 1	Maximum Shear Stress Ratio =	**0.055** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	6.375 k-ft	Va : Applied	2.250 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	Lr Only	Load Combination	Lr Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in Ratio =	0	<180	n/a
Max Upward Transient Deflection	0 in Ratio =	0	<180	n/a
Max Downward Total Deflection	0.098 in Ratio =	1107	>=120	Span: 1 : Lr Only
Max Upward Total Deflection	0 in Ratio =	0	<120	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 4.50 ft	1		0.000				32.70	19.58	1.00	1.00	-0.00	68.18	40.83
Lr Only													
Dsgn. L = 4.50 ft	1	0.326	0.055		-6.38	6.38	32.70	19.58	1.00	1.00	2.25	68.18	40.83
+0.750Lr													
Dsgn. L = 4.50 ft	1	0.244	0.041		-4.78	4.78	32.70	19.58	1.00	1.00	1.69	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Lr Only	1	0.0975	4.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.250	
Max Upward from Load Combinations	1.688	
Max Upward from Load Cases	2.250	
Lr Only	2.250	
+0.750Lr	1.688	

Steel Column

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Bike Rack Column (Strong Axis)

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS10x4x1/4**

Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	10.0 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 10.0 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 10.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 224.20 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.058, S = 4.232 k
BENDING LOADS . . .
Lat. Point Load at 10.0 ft creating Mx-x, E = 1.740 k
Moment acting about X-X axis at 10.0 ft, D = -0.5130, LR = -2.567 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.3062 : 1
	Load Combination	+D+0.70E
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	1.282 k
	Pn / Omega : Allowable	42.205 k
	Ma-x : Applied	-12.693 k-ft
	Mn-x / Omega : Allowable	43.613 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	22.954 k-ft
PASS	Maximum Shear Stress Ratio	0.0170 : 1
	Load Combination	+0.60D+0.70E
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	1.218 k
	Vn / Omega : Allowable	71.632 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	1.740 k

Maximum Load Deflections . . .

Along Y-Y 0.4603 in at 10.0ft above base
for load combination : E Only

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.027	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+D+Lr	0.086	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+D+S	0.077	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+D+0.750Lr	0.071	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+D+0.750S	0.065	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+0.60D	0.016	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.000	PASS	0.00 ft
+D+0.70E	0.306	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.017	PASS	0.00 ft
+D+0.750S+0.5250E	0.274	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.013	PASS	0.00 ft
+0.60D+0.70E	0.295	PASS	0.00 ft	1.63	1.00	72.41	148.24	0.017	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments (k-ft) @ Base	Mx - End Moments (k-ft) @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	1.282					-0.513			
+D+Lr	1.282					-3.080			
+D+S	5.514					-0.513			
+D+0.750Lr	1.282					-2.438			

Steel Column

DESCRIPTION: Bike Rack Column (Strong Axis)

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
		k				k-ft			
+D+0.750S	4.456					-0.513			
+0.60D	0.769					-0.308			
+D+0.70E	1.282			1.218		-12.693			
+D+0.750S+0.5250E	4.456			0.914		-9.648			
+0.60D+0.70E	0.769			1.218		-12.488			
Lr Only						-2.567			
S Only	4.232								
E Only				1.740		-17.400			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
			k				k-ft			
Axial @ Base	Maximum	5.514					-0.513			
"	Minimum						-2.567			
Reaction, X-X Axis Base	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			
Reaction, Y-Y Axis Base	Maximum				1.740		-17.400			
"	Minimum	1.282							-0.513	
Reaction, X-X Axis Top	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			
Reaction, Y-Y Axis Top	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			
Moment, X-X Axis Base	Maximum	4.232								
"	Minimum		-17.400		1.740		-17.400			
Moment, Y-Y Axis Base	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			
Moment, X-X Axis Top	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			
Moment, Y-Y Axis Top	Maximum	1.282					-0.513			
"	Minimum	1.282					-0.513			

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.020 in	10.000 ft
+D+Lr	0.0000 in	0.000 ft	0.122 in	10.000 ft
+D+S	0.0000 in	0.000 ft	0.020 in	10.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.097 in	10.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.020 in	10.000 ft
+0.60D	0.0000 in	0.000 ft	0.012 in	10.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.343 in	10.000 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	0.262 in	10.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.334 in	10.000 ft
Lr Only	0.0000 in	0.000 ft	0.102 in	10.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.456 in	9.933 ft

Steel Section Properties : HSS10x4x1/4

Steel Section Properties : HSS10x4x1/4

Steel Column

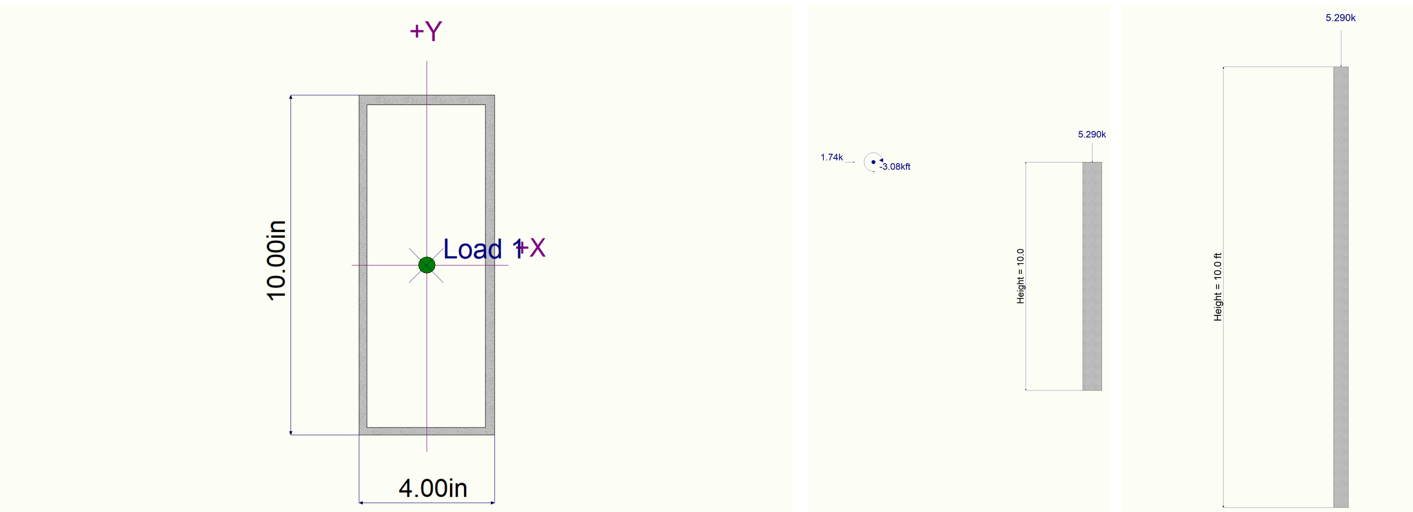
Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC.

DESCRIPTION: Bike Rack Column (Strong Axis)

Depth	=	10.000	in	I xx	=	74.70	in^4	J	=	47.400	in^4
Design Thick	=	0.233	in	S xx	=	14.90	in^3	Cw	=	17.10	in^6
Width	=	4.000	in	R xx	=	3.480	in				
Wall Thick	=	0.250	in	Zx	=	19.000	in^3				
Area	=	6.170	in^2	I yy	=	17.700	in^4	C	=	17.100	in^3
Weight	=	22.420	plf	S yy	=	8.870	in^3				
				R yy	=	1.700	in				
				Zy	=	10.000	in^3				
Ycg	=	0.000	in								

Sketches



Project Title:	USU Bike Canopy HYPR
Engineer:	
Project ID:	
Project Descr:	

Steel Column

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC.

DESCRIPTION: Bike Rack Column (Weak Axis)

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name :	**HSS10x4x1/4**	Overall Column Height	10.0 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top Free, Bottom Fixed
Steel Stress Grade		Brace condition :	
Fy : Steel Yield	46.0 ksi		
E : Elastic Bending Modulus	29,000.0 ksi		

Unbraced Length for buckling ABOUT X-X Axis = 10.0 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 10.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 224.20 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.058, S = 4.232 k
BENDING LOADS . . .
Lat. Point Load at 10.0 ft creating My-y, E = 1.742 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.5464	: 1
Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	1.282	k
Pn / Omega : Allowabl	42.205	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	43.613	k-ft
Ma-y : Applied	-12.194	k-ft
Mn-y / Omega : Allowable	22.954	k-ft
PASS Maximum Shear Stress Rati	0.04797	: 1
Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	1.219	k
Vn / Omega : Allowable	25.423	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	1.742 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 1.945 in at 10.0ft above base
for load combination : E Only

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.030	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.000	PASS	0.00 ft
+D+S	0.131	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.000	PASS	0.00 ft
+D+0.750S	0.106	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.000	PASS	0.00 ft
+0.60D	0.018	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.000	PASS	0.00 ft
+D+0.70E	0.546	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.048	PASS	0.00 ft
+D+0.750S+0.5250E	0.451	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.036	PASS	0.00 ft
+0.60D+0.70E	0.540	PASS	0.00 ft	1.00	1.67	72.41	148.24	0.048	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	1.282								
+D+S	5.514								
+D+0.750S	4.456								
+0.60D	0.769								
+D+0.70E	1.282	-1.219						-12.194	
+D+0.750S+0.5250E	4.456	-0.915						-9.146	
+0.60D+0.70E	0.769	-1.219						-12.194	

Steel Column

Project File: USU HYPR Bike Rack.ec6

DESCRIPTION: Bike Rack Column (Weak Axis)

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
S Only	4.232								
E Only		-1.742						-17.420	

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	5.514								
"	Minimum		-1.742						-17.420	
Reaction, X-X Axis Base	Maximum	1.282								
"	Minimum		-1.742						-17.420	
Reaction, Y-Y Axis Base	Maximum	1.282								
"	Minimum	1.282								
Reaction, X-X Axis Top	Maximum	1.282								
"	Minimum	1.282								
Reaction, Y-Y Axis Top	Maximum	1.282								
"	Minimum		-1.742						-17.420	
Moment, X-X Axis Base	Maximum	1.282								
"	Minimum	1.282								
Moment, Y-Y Axis Base	Maximum	1.282								
"	Minimum		-1.742						-17.420	
Moment, X-X Axis Top	Maximum	1.282								
"	Minimum	1.282								
Moment, Y-Y Axis Top	Maximum	1.282								
"	Minimum	1.282								

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	1.3614 in	10.000 ft	0.000 in	0.000 ft
+D+0.750S+0.5250E	1.0211 in	10.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	1.3614 in	10.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	1.9252 in	9.933 ft	0.000 in	0.000 ft

Steel Section Properties : HSS10x4x1/4

Depth	=	10.000 in	I xx	=	74.70 in^4	J	=	47.400 in^4
Design Thick	=	0.233 in	S xx	=	14.90 in^3	Cw	=	17.10 in^6
Width	=	4.000 in	R xx	=	3.480 in			
Wall Thick	=	0.250 in	Zx	=	19.000 in^3			
Area	=	6.170 in^2	I yy	=	17.700 in^4	C	=	17.100 in^3
Weight	=	22.420 plf	S yy	=	8.870 in^3			
			R yy	=	1.700 in			
			Zy	=	10.000 in^3			

Ycg	=	0.000 in

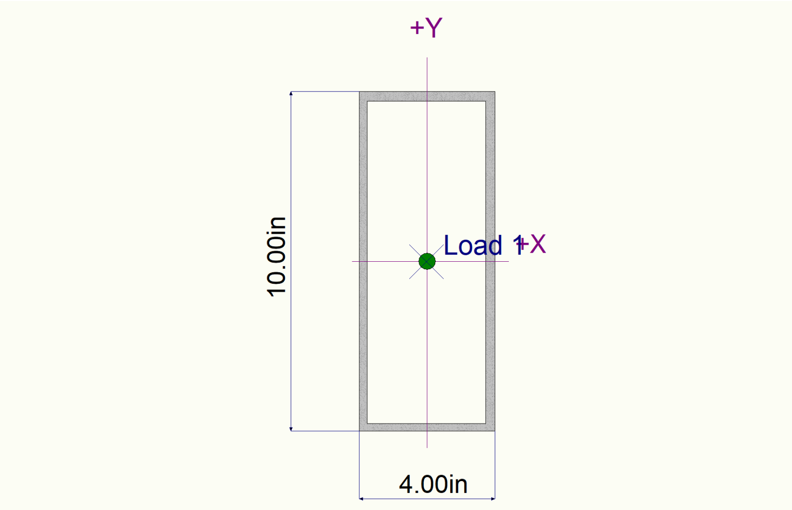
Steel Column

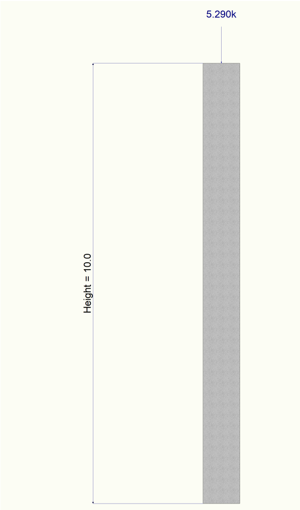
Project File: USU HYPR Bike Rack.ec6

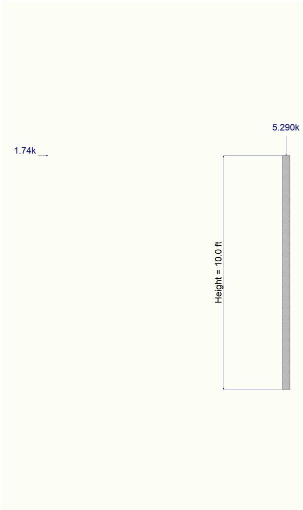
LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

DESCRIPTION: Bike Rack Column (Weak Axis)

Sketches







Pole Footing Embedded in Soil

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

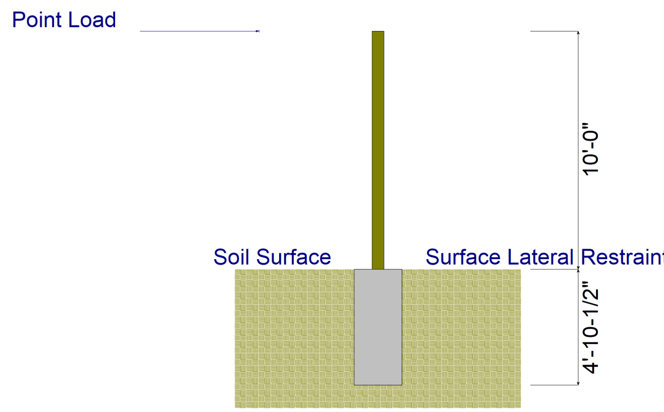
DESCRIPTION: Post (Strong Axis)

Code References

Calculations per IBC 2021 1807.3, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	24.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf



Controlling Values

Governing Load Combination	+D+0.70E
Lateral Load	1.219 k
Moment	12.723 k-ft

Restraint @ Ground Surface

Pressure at Depth	
Actual	**1,137.63** psf
Allowable	**1,218.75** psf
Surface Retraint Force	6,765.32 lbs
Minimum Required Depth	**4.875 ft**
Footing Base Area	3.142 ft^2
Maximum Soil Pressure	404.845 ksf

Applied Loads

	Lateral Concentrated Load (k)		Lateral Distributed Loads (k	Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k		k/ft	0.5290 k-ft	1.058 k
Lr : Roof Live	k		k/ft	k-ft	k
L : Live	k		k/ft	k-ft	k
S : Snow	k		k/ft	2.116 k-ft	4.232 k
W : Wind	0.3530 k		k/ft	k-ft	2,118.0 k
E : Earthquake	1.742 k		k/ft	k-ft	k
H : Lateral Earth	k		k/ft	k-ft	k
Load distance above ground surface	10.0 ft	TOP of Load above ground surface	ft		
		BOTTOM of Load above ground surface	ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.529	1.75	367.1	437.5	1.000
+D+S	0.000	2.645	2.88	680.0	718.8	1.000
+D+0.750S	0.000	2.116	2.63	652.6	656.3	1.000
+D+0.60W	0.212	2.647	2.88	680.5	718.8	1.000
+D+0.450W	0.159	2.118	2.63	653.0	656.3	1.000
+D+0.750S+0.450W	0.159	3.705	3.25	745.3	812.5	1.000
+0.60D+0.60W	0.212	2.435	2.75	684.3	687.5	1.000
+D+0.70E	1.219	12.723	4.88	1,137.6	1,218.8	1.000
+D+0.750S+0.5250E	0.915	11.262	4.63	1,118.7	1,156.3	1.000

DESCRIPTION: Post (Strong Axis)

+0.60D+0.70E	1.219	12.511	4.75	1,178.4	1,187.5	1.000

Concrete Column

Project File: USU HYPR Bike Rack.ec6

LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC.

DESCRIPTION: Foundation Pier For Canopy

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

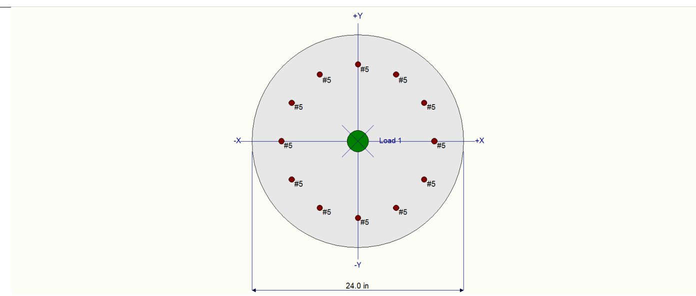
f'c : Concrete 28 day streng	=	3.0 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %
Seismic Design Category	=	A

Overall Column Height = 5.0 ft
End Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along colun
X-X (width) axis :
Unbraced Length for buckling ABOUT X-X Axis = 5.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 5.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 24.0in Diameter, Column Edge to Rebar Edge Cover = 3.0in

Column Reinforcing : 12 - #5 bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 2,356.19 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 5.0 ft above base, D = 1.058, S = 4.232, W = 2.118 k
BENDING LOADS . . .
Moment acting about X-X axisat 5.0 ft, W = 3.056, E = 16.0 k-ft
Lat. Point Load at 5.0 ft creating Mx-x, W = 0.4450, E = 1.375 k

DESIGN SUMMARY

Load Combination	+0.90D+E
Location of max.above base	4.966 ft
Maximum Stress Ratio	**0.097 : 1**

Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	3.073 k	φ * Pn =	31.987 k
Mu-x =	15.954 k-ft	φ * Mn-x =	166.010 k-ft
Mu-y =	0.3380 k-ft	φ * Mn-y =	2.911 k-ft
Mu Angle =	1.0 deg	φ =	0.90
Mu at Angle =	15.957 k-ft	φMn at Angle =	164.432 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . .

Pnmax : Nominal Max. Compressive Axial Capaci	1,367.31 k
Pnmin : Nominal Min. Tension Axial Capacity	k
φ Pn, max : Usable Compressive Axial Capacity	711.0 k
φ Pn, min : Usable Tension Axial Capacity	k

Maximum SERVICE Load Reactions .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	1.375 k

Maximum SERVICE Load Deflections . .

Along Y-Y -0.004814 in at 5.0 ft above base
for load combination : E Only
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Informatio β = 0.850 θ = 0.80

ρ : % Reinforcing	0.8223 %	Rebar < Min of 1.0 %
Reinforcing Area	3.720 in^2	
Concrete Area	452.389 in^2	

Concrete Column

DESCRIPTION: Foundation Pier For Canopy

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D	Actual	M2,min	4.97	4.78	711.00	1.000		1.000	0.53	90.000	0.53	78.54	**0.007**
+1.40D	M2,min	Actual	4.97	4.78	711.00	1.000	0.53	1.000		0.000	0.53	78.54	**0.007**
+1.20D	Actual	M2,min	4.97	4.10	711.00	1.000		1.000	0.45	90.000	0.45	78.54	**0.006**
+1.20D	M2,min	Actual	4.97	4.10	711.00	1.000	0.45	1.000		0.000	0.45	78.54	**0.006**
+1.20D+0.50S	Actual	M2,min	4.97	6.21	711.00	1.000		1.000	0.68	90.000	0.68	78.54	**0.009**
+1.20D+0.50S	M2,min	Actual	4.97	6.21	711.00	1.000	0.68	1.000		0.000	0.68	78.54	**0.009**
+1.20D+0.50W	Actual	M2,min	4.97	5.16	558.68	1.000	1.52	1.000	0.57	20.000	1.62	176.61	**0.009**
+1.20D+1.60S	Actual	M2,min	4.97	10.87	711.00	1.000		1.000	1.20	90.000	1.20	78.54	**0.015**
+1.20D+1.60S	M2,min	Actual	4.97	10.87	711.00	1.000	1.20	1.000		0.000	1.20	78.54	**0.015**
+1.20D+1.60S+0.50W	Actual	M2,min	4.97	11.93	703.97	1.000	1.52	1.000	1.31	41.000	2.01	119.73	**0.017**
+1.20D+W	Actual	M2,min	4.97	6.22	405.04	1.000	3.04	1.000	0.68	13.000	3.12	203.15	**0.015**
+1.20D+0.50S+W	Actual	M2,min	4.97	8.33	500.63	1.000	3.04	1.000	0.92	17.000	3.18	190.08	**0.017**
+0.90D+W	Actual	M2,min	4.97	5.19	346.94	1.000	3.04	1.000	0.57	11.000	3.09	206.65	**0.015**
+1.20D+0.20S+E	Actual	M2,min	4.97	4.94	54.86	1.000	15.95	1.000	0.54	2.000	15.96	177.11	**0.090**
+0.90D+E	Actual	M2,min	4.97	3.07	31.99	1.000	15.95	1.000	0.34	1.000	15.96	164.43	**0.097**

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only					3.414				
+D+S					7.646				
+D+0.750S					6.588				
+D+0.60W			0.267		4.685	-0.499			
+D+0.450W			0.200		4.367	-0.374			
+D+0.750S+0.450W			0.200		7.541	-0.374			
+0.60D+0.60W			0.267		3.319	-0.499			
+D+0.70E			0.963		3.414	-6.388			
+D+0.750S+0.5250E			0.722		6.588	-4.791			
+0.60D+0.70E			0.963		2.049	-6.388			
S Only					4.232				
W Only			0.445		2.118	-0.831			
E Only			1.375			-9.125			

Maximum Moment Reactions

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top		Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top	
D Only			k-ft			k-ft
+D+S			k-ft			k-ft
+D+0.750S			k-ft			k-ft
+D+0.60W	-0.499		k-ft			k-ft
+D+0.450W	-0.374		k-ft			k-ft
+D+0.750S+0.450W	-0.374		k-ft			k-ft
+0.60D+0.60W	-0.499		k-ft			k-ft
+D+0.70E	-6.388		k-ft			k-ft
+D+0.750S+0.5250E	-4.791		k-ft			k-ft
+0.60D+0.70E	-6.388		k-ft			k-ft
S Only			k-ft			k-ft
W Only	-0.831		k-ft			k-ft
E Only	-9.125		k-ft			k-ft

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	-0.000 in	5.000 ft
+D+0.450W	0.0000 in	0.000 ft	-0.000 in	5.000 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	-0.000 in	5.000 ft

Concrete Column

Project File: USU HYPR Bike Rack.ec6

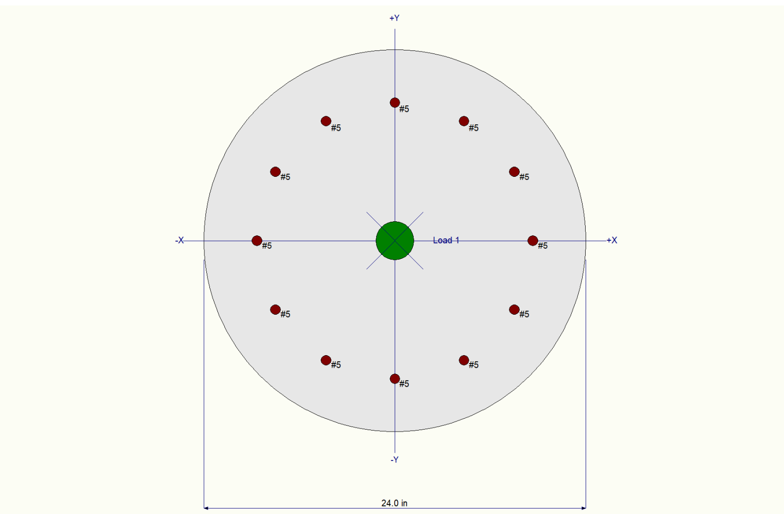
LIC# : KW-06016834, Build:20.23.07.20 FORSGREN ASSOCIATES, INC. (c) ENERCALC INC 1983-2023

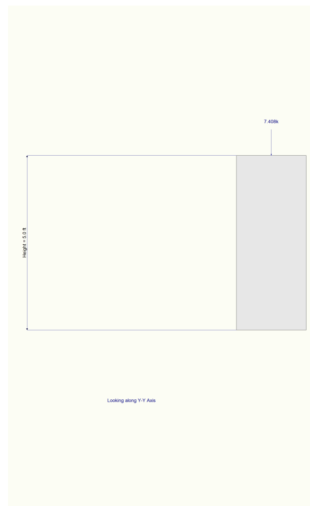
DESCRIPTION: Foundation Pier For Canopy

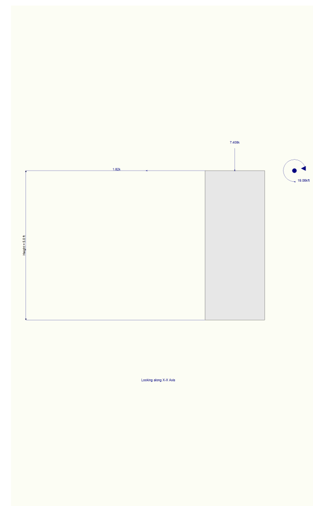
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+0.60D+0.60W	0.0000 in	0.000 ft	-0.000 in	5.000 ft
+D+0.70E	0.0000 in	0.000 ft	-0.003 in	5.000 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	-0.003 in	5.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	-0.003 in	5.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	-0.001 in	5.000 ft
E Only	0.0000 in	0.000 ft	-0.005 in	4.966 ft

Sketches







Interaction Diagrams

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Fastening point:			

Specifier's comments:

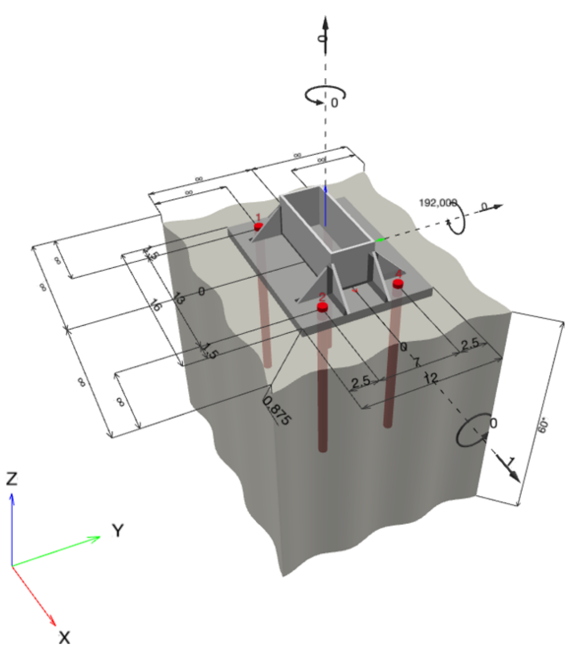
1 Anchor Design

1.1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 7/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 16.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.875 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 16.000 in. x 12.000 in. x 0.875 in.;
Profile:	Rectangular HSS (AISC), HSS10X4X.250; (L x W x T) = 10.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 60.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

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

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



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1.1.1 Design results

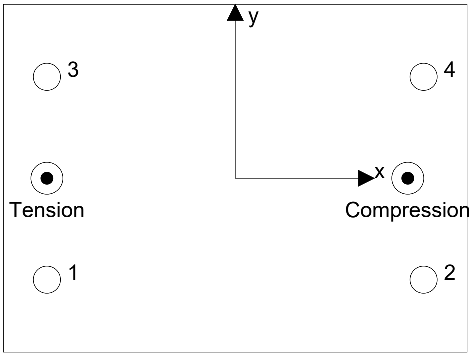
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 1; V_y = 0;$ $M_x = 0; M_y = 192{,}000; M_z = 0;$	no	40

1.2 Load case/Resulting anchor forces

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

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	7,906	39	9	38
2	-1	18	-8	-16
3	7,906	39	9	-38
4	-1	18	-8	16

resulting tension force in (x/y)=(-6.500/0.000): 15,810 [lb]
resulting compression force in (x/y)=(5.954/0.001): 15,957 [lb]



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

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	7,906	20,097	40	OK
Pullout Strength*	7,906	19,958	40	OK
Concrete Breakout Failure**	15,812	71,411	23	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

1.3.1 Steel Strength

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

N_{sa} [lb]
26,796

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
26,796	0.750	20,097	7,906

1.3.2 Pullout Strength

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.19	1.000	3,000

Calculations

N_p [lb]
28,512

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
28,512	0.700	19,958	7,906

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1.3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

$N_b = 16\ \lambda_a \sqrt{f_c'}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
16.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
-	16	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]
2,640.00	2,304.00	1.000	1.000	1.000	1.000	89,032

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
102,016	0.700	71,411	15,812

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1.4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	39	10,450	1	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1	180,325	1	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

1.4.1 Steel Strength

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

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

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

V_{sa} [lb]
16,078

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
16,078	0.650	10,450	39

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1.4.2 Pryout Strength

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

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

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

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

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

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

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

$N_b = 16\ \lambda_a \sqrt{f_c'}\ h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	16.000	0.000	0.157	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	∞	16	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]
3,355.00	2,304.00	1.000	0.994	1.000	1.000	89,032

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
257,607	0.700	180,325	1

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.396	0.004	5/3	22	OK

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

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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

1.7 Installation data

Profile: Rectangular HSS (AISC), HSS10X4X.250; (L x W x T) = 10.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 0.938 in.

Plate thickness (input): 0.875 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 7/8

Item number: not available

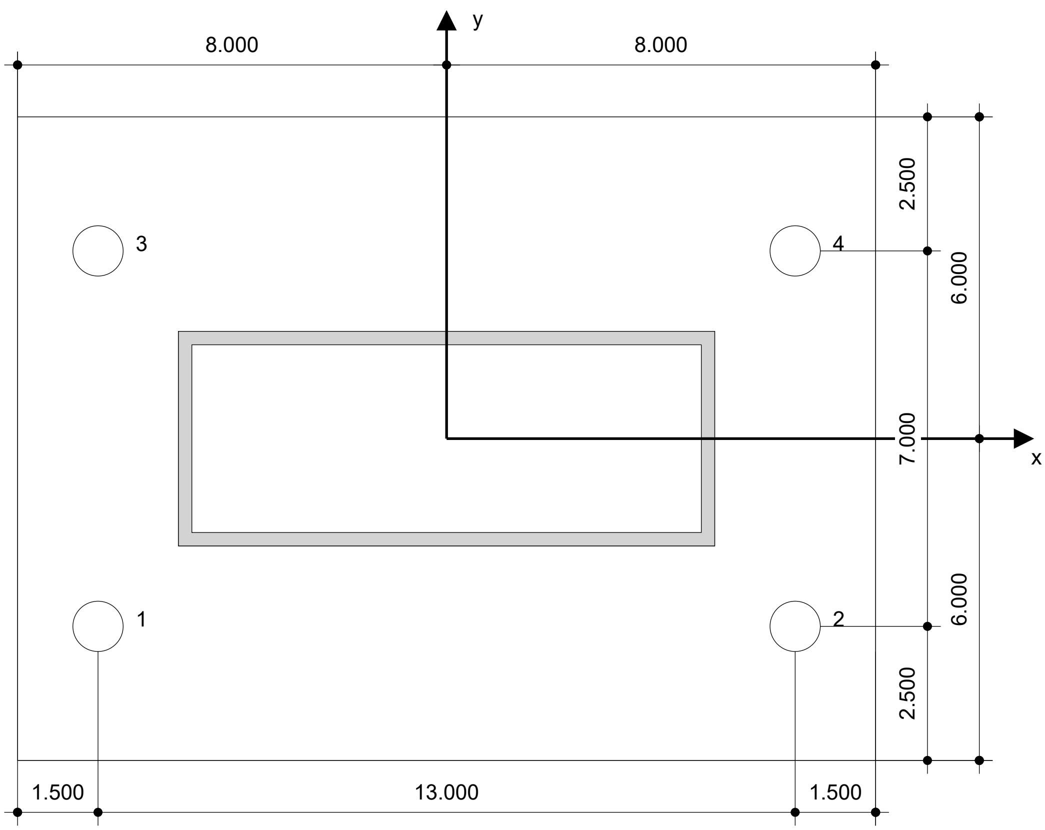
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 16.000 in.

Minimum thickness of the base material: 17.052 in.

Hilti Heavy Hex Head headed stud anchor with 16 in embedment, 7/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.500	-3.500	-	-	-	-
2	6.500	-3.500	-	-	-	-
3	-6.500	3.500	-	-	-	-
4	6.500	3.500	-	-	-	-

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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 16.000 in x 12.000 in x 0.875 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 7/8, h_{ef} = 16.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.875 in
Profile:	HSS10X4X.250; (L x W x T x FT) = 10.000 in x 4.000 in x 0.250 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 3000; $f_{c,cyl}$ = 3,000 psi; h = 60.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Stiffeners:	Geometry: Triangular; size = l_x x l_y x t = 3.000 in x 3.000 in x 0.250 in Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Welds (stiffeners to profile/anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Stiffeners		Anchor plate			Welds [%]	Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]		
1	Combination 1	12,957	0.00	26,763	0.00	10,454	0.00	1	36	7

2.3 Anchor plate classification

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

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	7,879 lb	7,906 lb
Anchor 2	-1 lb	-1 lb
Anchor 3	7,879 lb	7,906 lb
Anchor 4	-1 lb	-1 lb

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

2.4 Profile/Stiffeners/Plate

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

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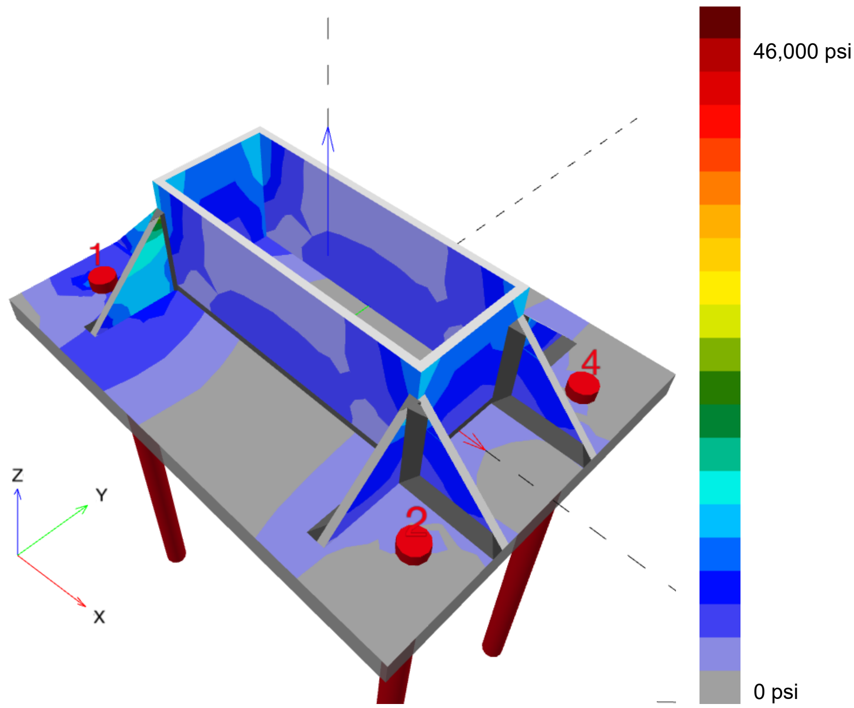
2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	10,454	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	12,957	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	11,714	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	11,689	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	11,693	0.00	OK
Stiffener	Combination 1	ASTM A36	36,000	5.00	26,763	0.00	OK

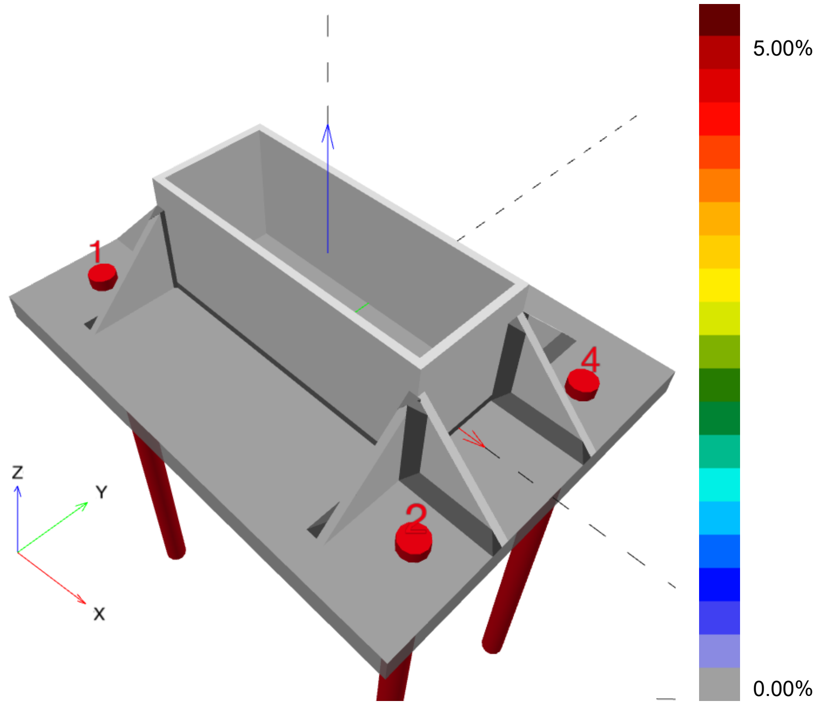
2.4.1.1 Equivalent stress

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



2.4.1.2 Plastic strain

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



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

Decisive load combination: 1 - Combination 1

Equations

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

$\Phi R_n = 0.75\ R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	2.094	0.875	58,000	0.875	106,599
Anchor 2	2.831	0.875	58,000	0.875	106,599
Anchor 3	2.094	0.875	58,000	0.875	106,599
Anchor 4	2.831	0.875	58,000	0.875	106,599

Results

	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	39	79,949	1	OK
Anchor 2	18	79,949	1	OK
Anchor 3	39	79,949	1	OK

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	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 4	18	79,949	1	OK

2.5 Welds

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

2.5.1 Stiffeners to profile/anchor plate

Decisive load combination: 1 - Combination 1

Equations

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

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

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

Variables

Edge	X_u	T_h [in]	L_s [in]	L [in]	L_c [in]	F_{EXX} [psi]	Θ [°]	A_w [in²]
Stiffenera	E70xx	◢0.250	0.354	2.969	0.371	70,000	40.8	0.09
Stiffenerb	E70xx	◢0.250	0.354	2.969	0.371	70,000	40.1	0.09
Stiffenerc	E70xx	◢0.250	0.354	2.969	0.371	70,000	48.0	0.09
Stiffenerd	E70xx	◢0.250	0.354	2.969	0.371	70,000	44.3	0.09
Stiffenere (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	48.9	0.09
Stiffenerf (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	48.0	0.09
Stiffenerg (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	49.0	0.09
Stiffenerh (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	48.1	0.09
Stiffenere (Member 1-bfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	24.2	0.09
Stiffenerf (Member 1-tfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	25.8	0.09
Stiffenerg (Member 1-bfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	24.2	0.09
Stiffenerh (Member 1-tfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	25.8	0.09

Results

Edge	F_n [lb]	ΦR_n [lb]	Utilization [%]	Status
Stiffenera	923	3,693	25	OK
Stiffenerb	946	3,677	26	OK
Stiffenerc	483	3,858	13	OK
Stiffenerd	484	3,775	13	OK
Stiffenere (Anchor plate)	537	3,879	14	OK

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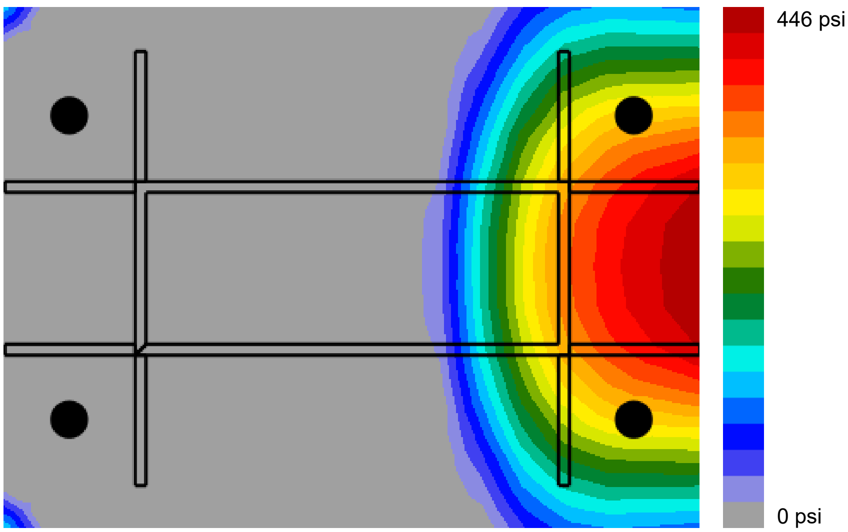
Edge	F_n [lb]	ΦR_n [lb]	Utilization [%]	Status
Stiffenerf (Anchor plate)	808	3,857	21	OK
Stiffenerg (Anchor plate)	537	3,879	14	OK
Stiffenerh (Anchor plate)	809	3,861	21	OK
Stiffenere (Member 1-bfl 1)	958	3,323	29	OK
Stiffenerf (Member 1-tfl 1)	1,187	3,359	36	OK
Stiffenerg (Member 1-bfl 1)	958	3,323	29	OK
Stiffenerh (Member 1-tfl 1)	1,188	3,358	36	OK

2.6 Concrete

Decisive load combination: 1 - Combination 1

2.6.1 Compression in concrete under the anchor plate



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

Equations

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

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

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

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

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Variables

N [lb]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
15,957	3,000	0.65	78.49	52,121.20

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,315	203	7	OK

2.7 Symbol explanation

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

2.8 Warnings

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

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3 Summary of results

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

	Load combination	Max. utilization	Status
Anchors	Combination 1	40%	OK
Anchor plate	Combination 1	30%	OK
Welds	Combination 1	36%	OK
Stiffeners	Combination 1	75%	OK
Concrete	Combination 1	7%	OK
Profile	Combination 1	29%	OK

Fastening meets the design criteria!

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4 Remarks; Your Cooperation Duties

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Specifier's comments:

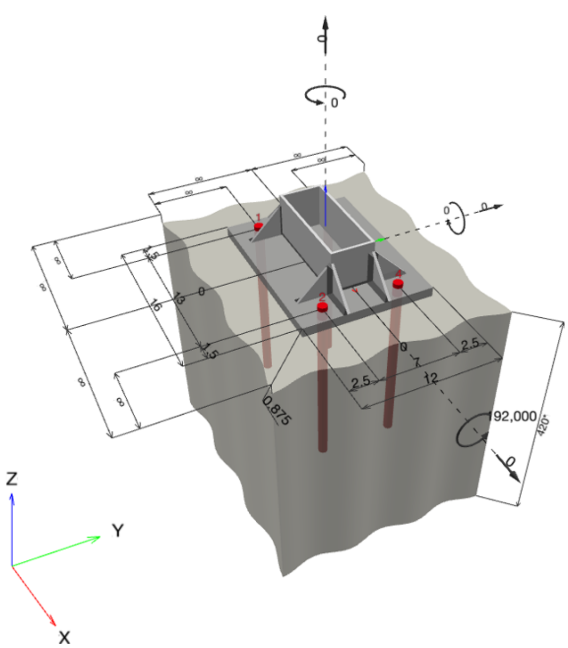
1 Anchor Design

1.1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 7/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 16.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.875 in.
Anchor plate^{CBFEM} :	l_x x l_y x t = 16.000 in. x 12.000 in. x 0.875 in.;
Profile:	Rectangular HSS (AISC), HSS10X4X.250; (L x W x T) = 10.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 420.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

^{CBFEM} - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

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



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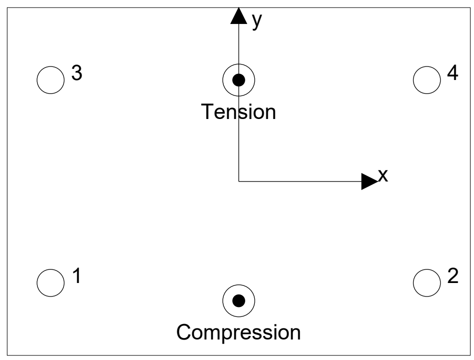
1.1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0$; $V_x = 0$; $V_y = 0$; $M_x = 192{,}000$; $M_y = 0$; $M_z = 0$;	no	65

1.2 Load case/Resulting anchor forces

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

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-1	22	-21	-3
2	-1	22	21	-4
3	12,797	135	135	4
4	12,793	135	-135	3



resulting tension force in (x/y)=(-0.001/3.500): 25,587 [lb]
resulting compression force in (x/y)=(0.001/-4.117): 25,873 [lb]

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

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	12,797	20,097	64	OK
Pullout Strength*	12,797	19,958	65	OK
Concrete Breakout Failure**	25,590	79,199	33	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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1.3.1 Steel Strength

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

N_{sa} [lb]
26,796

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
26,796	0.750	20,097	12,797

1.3.2 Pullout Strength

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.19	1.000	3,000

Calculations

N_p [lb]
28,512

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
28,512	0.700	19,958	12,797

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1.3.3 Concrete Breakout Failure

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

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

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

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

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

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — 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)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
16.000	0.001	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	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]
2,928.00	2,304.00	1.000	1.000	1.000	1.000	89,032

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
113,141	0.700	79,199	25,590

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1.4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	135	10,450	2	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	135	45,376	1	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

1.4.1 Steel Strength

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

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

V_{sa} [lb]
16,078

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
16,078	0.650	10,450	135

1.4.2 Pryout Strength

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

$\phi\ V_{cp} \geq V_{ua}$ 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)

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

$N_b = 16\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	16.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	∞	16	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]
838.75	2,304.00	1.000	1.000	1.000	1.000	89,032

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
64,823	0.700	45,376	135

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.641	0.013	5/3	48	OK

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

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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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1.7 Installation data

Profile: Rectangular HSS (AISC), HSS10X4X.250; (L x W x T) = 10.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 0.938 in.

Plate thickness (input): 0.875 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 7/8

Item number: not available

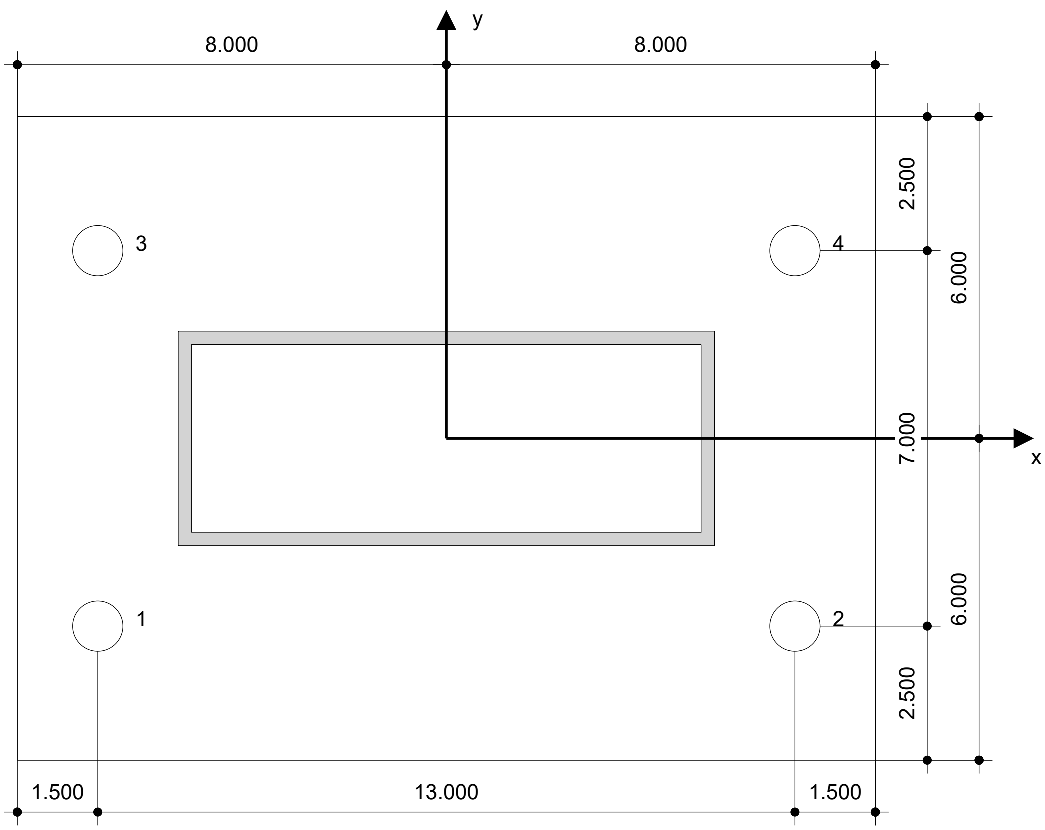
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 16.000 in.

Minimum thickness of the base material: 17.052 in.

Hilti Heavy Hex Head headed stud anchor with 16 in embedment, 7/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.500	-3.500	-	-	-	-
2	6.500	-3.500	-	-	-	-
3	-6.500	3.500	-	-	-	-
4	6.500	3.500	-	-	-	-

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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 16.000 in x 12.000 in x 0.875 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 7/8, h_{ef} = 16.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.875 in
Profile:	HSS10X4X.250; (L x W x T x FT) = 10.000 in x 4.000 in x 0.250 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 3000; $f_{c,cyl}$ = 3,000 psi; h = 420.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Stiffeners:	Geometry: Triangular; size = l_x x l_y x t = 3.000 in x 3.000 in x 0.250 in Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Welds (stiffeners to profile/anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Stiffeners		Anchor plate			Welds [%]	Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]		
1	Combination 1	33,788	0.00	36,039	0.14	19,769	0.00	1	71	10

2.3 Anchor plate classification

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

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-1 lb	-1 lb
Anchor 2	-1 lb	-1 lb
Anchor 3	12,145 lb	12,797 lb
Anchor 4	12,145 lb	12,793 lb

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

2.4 Profile/Stiffeners/Plate

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

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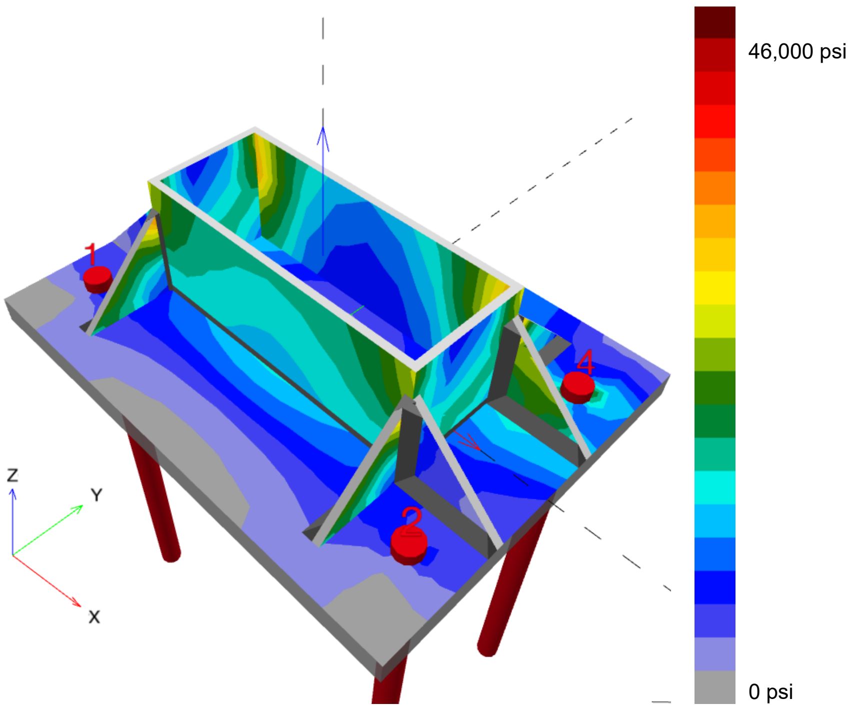
2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	19,769	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	22,015	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	22,008	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	24,747	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	33,788	0.00	OK
Stiffener	Combination 1	ASTM A36	36,000	5.00	36,039	0.14	OK

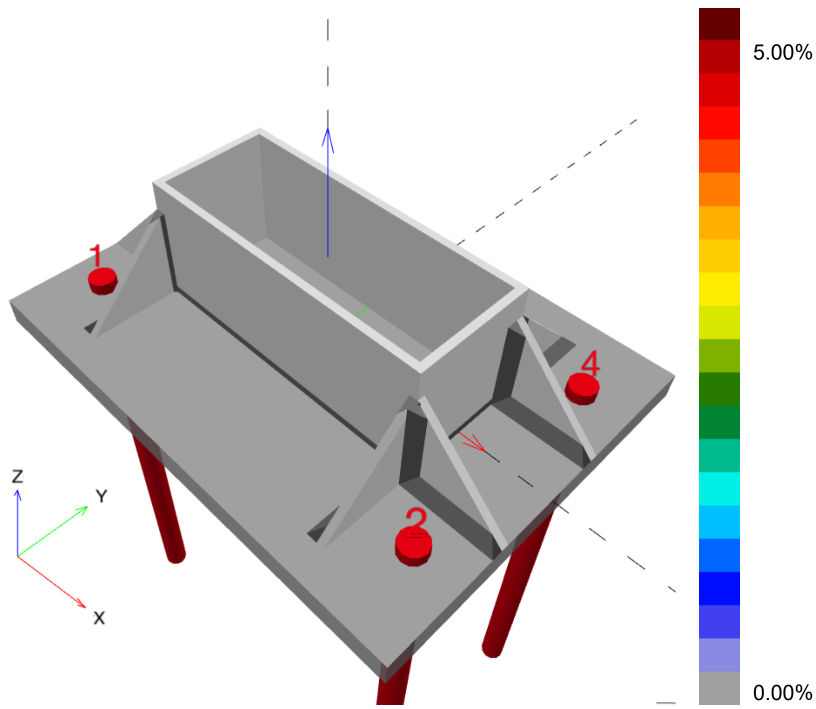
2.4.1.1 Equivalent stress

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



2.4.1.2 Plastic strain

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



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

Decisive load combination: 1 - Combination 1

Equations

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

ΦR_n = $0.75\ R_n$

V $\leq$ ΦR_n

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	14.224	0.875	58,000	0.875	106,599
Anchor 2	14.237	0.875	58,000	0.875	106,599
Anchor 3	1.032	0.875	58,000	0.875	62,850
Anchor 4	1.032	0.875	58,000	0.875	62,848

Results

	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 1	22	79,949	1	OK
Anchor 2	22	79,949	1	OK
Anchor 3	135	47,137	1	OK

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	V [lb]	ΦR_n [lb]	Utilization [%]	Status
Anchor 4	135	47,136	1	OK

2.5 Welds

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

2.5.1 Stiffeners to profile/anchor plate

Decisive load combination: 1 - Combination 1

Equations

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

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

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

Variables

Edge	X_u	T_h [in]	L_s [in]	L [in]	L_c [in]	F_{EXX} [psi]	Θ [°]	A_w [in²]
Stiffenera	E70xx	◢0.250	0.354	2.969	0.371	70,000	43.9	0.09
Stiffenerb	E70xx	◢0.250	0.354	2.969	0.371	70,000	34.9	0.09
Stiffenerc	E70xx	◢0.250	0.354	2.969	0.371	70,000	43.9	0.09
Stiffenerd	E70xx	◢0.250	0.354	2.969	0.371	70,000	36.2	0.09
Stiffenere (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	61.4	0.09
Stifenerf (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	61.8	0.09
Stiffenerg (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	42.1	0.09
Stiffenerh (Anchor plate)	E70xx	◢0.250	0.354	2.969	0.371	70,000	42.2	0.09
Stiffenere (Member 1-bfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	18.3	0.09
Stiffenerf (Member 1-tfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	18.3	0.09
Stiffenerg (Member 1-bfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	27.2	0.09
Stiffenerh (Member 1-tfl 1)	E70xx	◢0.250	0.354	2.984	0.373	70,000	27.3	0.09

Results

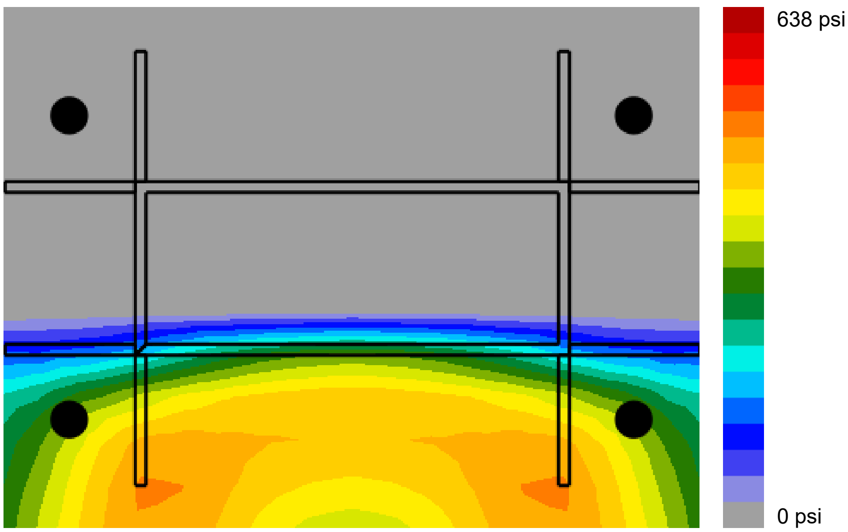
Edge	F_n [lb]	ΦR_n [lb]	Utilization [%]	Status
Stiffenera	1,757	3,765	47	OK
Stiffenerb	1,201	3,555	34	OK
Stiffenerc	1,756	3,766	47	OK
Stiffenerd	1,159	3,585	33	OK
Stiffenere (Anchor plate)	551	4,124	14	OK

Edge	F_n [lb]	ΦR_n [lb]	Utilization [%]	Status
Stiffenerf (Anchor plate)	550	4,131	14	OK
Stiffenerg (Anchor plate)	1,703	3,724	46	OK
Stiffenerh (Anchor plate)	1,696	3,727	46	OK
Stiffenere (Member 1-bfl 1)	1,226	3,197	39	OK
Stiffenerf (Member 1-tfl 1)	1,226	3,197	39	OK
Stiffenerg (Member 1-bfl 1)	2,374	3,391	71	OK
Stiffenerh (Member 1-tfl 1)	2,370	3,393	70	OK

2.6 Concrete

Decisive load combination: 1 - Combination 1

2.6.1 Compression in concrete under the anchor plate



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

Equations

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

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

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

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

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Variables

N [lb]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
25,873	3,000	0.65	80.12	530,435.23

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,315	323	10	OK

2.7 Symbol explanation

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

2.8 Warnings

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

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3 Summary of results

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

	Load combination	Max. utilization	Status
Anchors	Combination 1	65%	OK
Anchor plate	Combination 1	55%	OK
Welds	Combination 1	71%	OK
Stiffeners	Combination 1	100%	OK
Concrete	Combination 1	10%	OK
Profile	Combination 1	74%	OK

Fastening meets the design criteria!

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4 Remarks; Your Cooperation Duties

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TABLE 1—HOLLO-BOLT® BLIND FASTENER TECHNICAL DATA

PART NUMBER & DESCRIPTION			DIMENSIONAL INFORMATION[3] (inches)							TECHNICAL		ALLOWABLE LOADING[2] STATIC AND SDC A, B & C ONLY				ALLOWABLE LOADING[2] STATIC AND SDC D, E & F ONLY			
												LRFD Method		ASD Method		LRFD Method		ASD Method	
Hollo-Bolt Part Number	Hollo-Bolt (Core Bolt Size)	Description	Core Bolt Length	Clamping Range Dim W Min[1]	Clamping Range Dim W Max	Across Flats of Collar A/F	Collar Thickness H	Dim A	Dim B	Dim d_1 Drill Diameter (Inches)	Torque (Ft lb)	Tensile lbs	Shear lbs	Tensile lbs	Shear lbs	Tensile lbs	Shear lbs	Tensile lbs	Shear lbs
LHBM08#1	$^{5}/_{16}$	$^{5}/_{16}$" Hollo-Bolt Size 1	2	$^{1}/_{4}$	$^{7}/_{8}$	$^{3}/_{4}$	$^{3}/_{16}$	$1^{3}/_{8}$	$^{1}/_{2}$	$^{9}/_{16}$	17	3775	3215	2340	2000	3305	2675	2045	1665
LHBM08#2	$^{5}/_{16}$	$^{5}/_{16}$" Hollo-Bolt Size 2	$2^{3}/_{4}$	$^{7}/_{8}$	$1^{5}/_{8}$	$^{3}/_{4}$	$^{3}/_{16}$	$1^{3}/_{8}$	$^{1}/_{2}$	$^{9}/_{16}$	17	3775	3215	2340	2000	3305	2675	2045	1665
LHBM08#3	$^{5}/_{16}$	$^{5}/_{16}$" Hollo-Bolt Size 3	$3^{5}/_{8}$	$1^{5}/_{8}$	$2^{3}/_{8}$	$^{3}/_{4}$	$^{3}/_{16}$	$1^{3}/_{8}$	$^{1}/_{2}$	$^{9}/_{16}$	17	3775	3215	2340	2000	3305	2675	2045	1665
LHBM10#1	$^{3}/_{8}$	$^{3}/_{8}$" Hollo-Bolt Size 1	$2^{1}/_{4}$	$^{5}/_{16}$	$^{7}/_{8}$	$^{15}/_{16}$	$^{1}/_{4}$	$1^{9}/_{16}$	$^{9}/_{16}$	$^{3}/_{4}$	33	6160	5485	3820	3415	5485	4565	3395	2830
LHBM10#2	$^{3}/_{8}$	$^{3}/_{8}$" Hollo-Bolt Size 2	3	$^{7}/_{8}$	$1^{5}/_{8}$	$^{15}/_{16}$	$^{1}/_{4}$	$1^{9}/_{16}$	$^{9}/_{16}$	$^{3}/_{4}$	33	6160	5485	3820	3415	5485	4565	3395	2830
LHBM10#3	$^{3}/_{8}$	$^{3}/_{8}$" Hollo-Bolt Size 3	$3^{5}/_{8}$	$1^{5}/_{8}$	$2^{3}/_{8}$	$^{15}/_{16}$	$^{1}/_{4}$	$1^{9}/_{16}$	$^{9}/_{16}$	$^{3}/_{4}$	33	6160	5485	3820	3415	5485	4565	3395	2830
LHBM12#1	$^{1}/_{2}$	$^{1}/_{2}$" Hollo-Bolt Size 1	$2^{3}/_{8}$	$^{5}/_{16}$	1	$1^{3}/_{16}$	$^{1}/_{4}$	2	$^{3}/_{4}$	$^{13}/_{16}$	59	8545	7485	5305	4675	7465	6250	4630	3890
LHBM12#2	$^{1}/_{2}$	$^{1}/_{2}$" Hollo-Bolt Size 2	$3^{5}/_{8}$	1	$1^{13}/_{16}$	$1^{3}/_{16}$	$^{1}/_{4}$	2	$^{3}/_{4}$	$^{13}/_{16}$	59	8545	7485	5305	4675	7465	6250	4630	3890
LHBM12#3	$^{1}/_{2}$	$^{1}/_{2}$" Hollo-Bolt Size 3	$4^{1}/_{4}$	$1^{13}/_{16}$	$2^{3}/_{4}$	$1^{3}/_{16}$	$^{1}/_{4}$	2	$^{3}/_{4}$	$^{13}/_{16}$	59	8545	7485	5305	4675	7465	6250	4630	3890
LHBM16#1	$^{5}/_{8}$	$^{5}/_{8}$" Hollo-Bolt Size 1	3	$^{1}/_{2}$	$1^{1}/_{8}$	$1^{3}/_{8}$	$^{5}/_{16}$	$2^{3}/_{16}$	$^{13}/_{16}$	$1^{1}/_{16}$	140	13915	11645	8635	7285	13330	9780	8270	6090
LHBM16#2	$^{5}/_{8}$	$^{5}/_{8}$" Hollo-Bolt Size 2	4	$1^{1}/_{8}$	2	$1^{3}/_{8}$	$^{5}/_{16}$	$2^{3}/_{16}$	$^{13}/_{16}$	$1^{1}/_{16}$	140	13915	11645	8635	7285	13330	9780	8270	6090
LHBM16#3	$^{5}/_{8}$	$^{5}/_{8}$" Hollo-Bolt Size 3	$4^{3}/_{4}$	2	$2^{13}/_{16}$	$1^{3}/_{8}$	$^{5}/_{16}$	$2^{3}/_{16}$	$^{13}/_{16}$	$1^{1}/_{16}$	140	13915	11645	8635	7285	13330	9780	8270	6090
LHBM20#1	$^{3}/_{4}$	$^{3}/_{4}$" Hollo-Bolt Size 1	$3^{5}/_{8}$	$^{1}/_{2}$	$1^{5}/_{16}$	$1^{13}/_{16}$	$^{3}/_{8}$	$2^{3}/_{4}$	1	$1^{5}/_{16}$	221	19985	18390	12410	11490	19355	15330	12005	9555
LHBM20#2	$^{3}/_{4}$	$^{3}/_{4}$" Hollo-Bolt Size 2	$4^{3}/_{4}$	$1^{5}/_{16}$	$2^{3}/_{8}$	$1^{13}/_{16}$	$^{3}/_{8}$	$2^{3}/_{4}$	1	$1^{5}/_{16}$	221	19985	18390	12410	11490	19355	15330	12005	9555
LHBM20#3	$^{3}/_{4}$	$^{3}/_{4}$" Hollo-Bolt Size 3	$5^{7}/_{8}$	$2^{3}/_{8}$	$3^{3}/_{8}$	$1^{13}/_{16}$	$^{3}/_{8}$	$2^{3}/_{4}$	1	$1^{5}/_{16}$	221	19985	18390	12410	11490	19355	15330	12005	9555

[1]The minimum clamping thickness specified is based on AC437 section 4.1.1

[2]From tests performed we have used the following lowest factors for the LRFD and ASD calculations:
Tensile: LRFD $\phi = 0.51$, ASD $\Omega = 3.16$
Shear: LRFD $\phi = 0.50$, ASD $\Omega = 3.21$
ASD Method is approximately equal to LRFD Method divided by 1.6

[3]See Figure 2 for additional information on dimensions.