Allister
Exterior Renovation
Mercer Island, Washington

Structural Calculations

CALCULATIONS INCLUDED:

Pages 3 through 27

These Calculations cover the design modifications the exterior of the Allister Restaurant.

ANDREW WAYNE EWING
STATE OF WASHINGTON
48649
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER
11/17/23

kpff
1601 5th Avenue, Suite 1600
Seattle, WA 98101 206.622.5822

KPFF Project No. 2300702

11/17/2023

DESIGN SUMMARY

Date: *11/17/2023*
By: *LJS*

Design: *Canopies*

This structural calculation package covers the modifications to the canopy at Allister located at 7650 SE 27th St #100, Mercer Island, WA. All structural designs follow the provisions of IBC 2018 and IEBC and their adopted codes.

New Canopies

The existing radial canopies at the southwestern corner of the building will be replaced with a new rectangular canopy. The new canopy will cover roughly the same area as the existing canopy. The existing canopy consists of glass on steel framing, utilizing guy wires and a bolted connection back to the existing structure. The new canopy will utilize a post installed plate connections at the existing building and new HSS posts away from the building to support the other side of the span. As result the existing building wall is carrying less vertical load than it does with the current canopies as the tributary area is roughly the same, but half the load will flow to the new HSS posts.

The canopy steel frame will be fully welded together. This will provide lateral resistance out-of-plane. In plane forces will be transferred back into the existing structure.

Notes / Assumptions:

- Loads between existing and new canopy are identical
    - 25 psf Snow, 20 psf Live,
    - new and existing are steel framed with glass >> similar dead loads.

- Existing canopy uses a guy wire system to support outer edge of canopy, imposing a force-couple back at the building. The new canopy condition is a simple span to new posts, so this force couple will be removed at the building. Therefore if the tributary area of canopy supported by the existing concrete wall remains roughly the same then there is no increase in the vertical force going into the existing wall.

Existing Canopy Properties

Tributary Width of Beam/Guy	7.17	feet
Canopy Length, L =	5.50	feet
Support/Guy Tributary Area	39.44	ft2
>> Full Triubtary into existing concrete wall		

New Canopy Properties

Tributary Width of Beam =	3.17	feet
Average Canopy Length, L =	13.33	feet
worst case (longest) canopy length used		
Trib Length into Existing Concrete Wall =	6.67	feet
total length / 2 for simple span beams		
New Canopy Beam Tributary Area =	21.1	ft2
>> Tributary area inot existing concrete wall		

Loads

Vertical Reactions into Existing Concrete Wall		
Controlling Load: from analysis	**64.0**	**psf**
Existing Canopy	2.52	kips
New Canopy	1.35	kips
Change in Vertical Reaction in Existing Wall:	-46%	

>> Total vertical reaction into existing wall decreases. No modifications required to existing structure per IEBC.

Loads

Dead Area Loads

Glass Canopy	5	psf
MEP Allowance	5	psf
Total	**10**	**psf**

Note: Beam selfweight accounted for in analysis program.

Live Area Load

Typical Roof Live Load	20	psf
Total	**20**	**psf**

Snow Area Loads

Typical Snow Load	25	psf
Total	**25**	**psf**

Wind Area Loads

Wind Downforce	18.8	psf
Wind Uplift	24.0	psf

See Meca Wind Summary for Results

Seismic Loads

- Canopy is less than 25% of combined structure weight, use ASCE 7-16 Chapter 13
- Assume *Other Rigid Components, Low Deformability* for Glass

Spectral Acceleration, Sds =	1.115				
Component Amplification Factor, ap =	1.00				
Component Response Factor, Rp =	1.50				
Overstrength Factor, omega_0 =	1.50				
Seismic Weight of Longest Beam + Trib Glass =	0.580	kips			
Importance Factor, Ip =	1.00				
Average Building Roof Height, h =	55	feet			
Component Attachement Height, z =	12.67	feet			
Calculated Comp Seismic Force, Fp =	0.252	kips			
Code Min Comp Seismic Force, Fp,min =	0.194	kips			
Code Max Comp Seismic Force, Fp,max =	1.03	kips			
Design Horizontal Seismic Force, Fp =	**0.252**	**kips**	**=**	**0.019**	**klf**
Design Vertical Seismic Force, Fpv =	**0.129**	**kips**	**=**	**0.010**	**klf**

>> Use above area loads in Enercalc Analysis

Beam Loads

Max Beam Tributary Width, b =	3.17	ft	*interior beams*
Max Beam Span, L =	13.33	ft	*western edge beam*

>> Use above beam properties in Enercalc Analysis

Analysis Output

>> Wind controls over seismic forces in the vertical direction, seismic controls connections lateral capacity
>> HSS6x2x1/4 is adequate for the above loading and meets serviceability requirements for glass.

>> See attached Enercalc Analysis Summary

Extreme Beam Reactions:

	Building Side		Header Side	
Dead	0.29	kips	0.29	kips
Live	0	kips	0	kips
Snow	0.57	kips	0.57	kips
Wind (downforce)	0.43	kips	0.43	kips
Wind (uplift)	-0.55	kips	-0.55	kips
Seismic		kips		kips

>> Use above reactions to design header beams at pergola

>> For both wind conditions, **HSS8x4x1/8** provides adequate strength and serviceability performance.

MecaWind v2452

Calculations Prepared by:
Date: Nov 14, 2023
File Location: C:\Users\LukeS\Desktop\2300XXX Allister\2023.11.14 Meca Wind Run.wnd

General:

Wind Load Standard	=	ASCE 7-16	Basic Wind Speed	=	110.0 mph
Exposure Classification	=	B	Risk Category	=	II
Structure Type	=	Canopy	Design Basis for Wind Pressures	=	LRFD
MWFRS Analysis Method	=	None	C&C Analysis Method	=	Ch 30 Pt 6
Dynamic Type of Structure	=	Rigid	Show Advanced Options	=	0

Building Inputs

h	=	Mean Building Roof Height	=	46.000 ft
he	=	Mean Eave Height	=	46.000 ft
hc	=	Mean canopy height	=	12.670 ft

Exposure Constants [Tbl 26.11-1]:

α = 3-s Gust-speed exponent	=	7.000	Z_g= Nominal Ht of Boundary Layer	=	1200.000 ft
$\hat{a}$ = Reciprocol of α	=	0.143	b = 3 sec gust speed factor	=	0.840
α_m= Mean hourly Wind-Speed Exponent	=	0.250	b_m= Mean hourly Windspeed Exponent	=	0.450
c = Turbulence Intensity Factor	=	0.300	ε = Integral Length Scale Exponent	=	0.3333

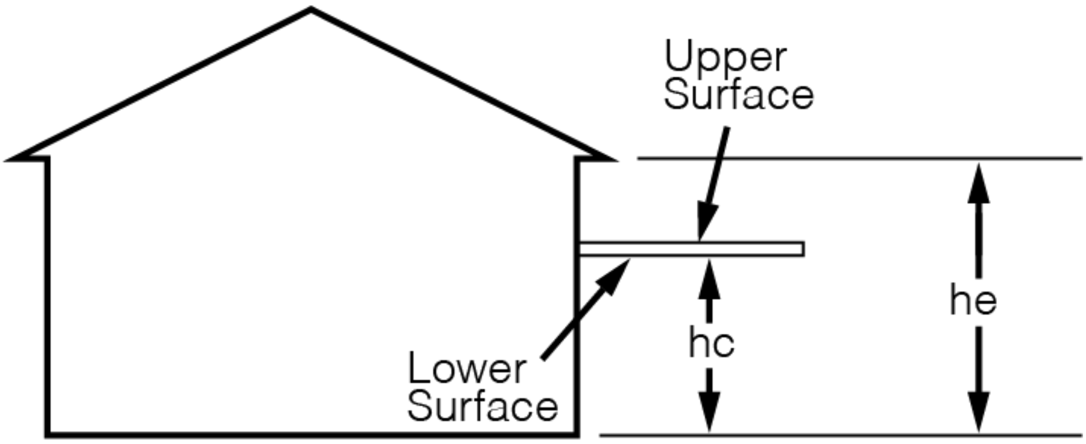
Components and Cladding (C&C) Canopy Wind Loads per Ch 30 Pt 6:

h	=	Mean structure height	=	46.000 ft
K_h	=	$2.01\bullet(h/Z_g)^{2/\alpha}$[Tbl 26.10-1]	=	0.792
K_{zt}	=	No Topographic feature specified	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85
LF	=	Load Factor based upon STRENGTH Design	=	1.00
K_e	=	Ground Elev Factor [Tbl 26.9-1]	=	1.000
q_h	=	$0.00256\bullet K_h\bullet K_{zt}\bullet K_d\bullet K_e\bullet V^2*LF$ [Eq 26.10-1]	=	20.84 psf

Description	Zone	Width ft	Span ft	Area ft²	Figure	GCp Pos	GCp Neg	P Min psf	P Max psf
Zone Lower Surface	Lower Surface	13.500	13.330	179.95	30.11-1A/B	0.600	-0.650	-16.00	16.00
Zone Upper Surface	Upper Surface	13.500	13.330	179.95	30.11-1A/B	0.600	-0.750	-16.00	16.00
Zone Upr & Lwr Surf	Upr & Lwr Surf	13.500	13.330	179.95	30.11-1A/B	0.650	-0.500	-16.00	16.00

Area = Span Length x Effective Width
GCp = External Pressure Coefficients
P = Wind Pressure: $q_h\bullet GC_p$[Eq 30.11-1]
Zone = 'Upr & Lwr Surf' is simultaneous contributions from both Upper & Lower
** Per § 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}*

Components and Cladding (C&C) Zone Summary per Ch 30 Pt 6:



Zone = "Upr & Lwr Surf" Considers Simultaneous contribution from both upper and lower surfaces.

Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 6
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	Pos A ≤ 10 ft² psf	Neg A ≤ 10 ft² psf	Pos A = 20 ft² psf	Neg A = 20 ft² psf	Pos A = 50 ft² psf	Neg A = 50 ft² psf	Pos A > 100 ft² psf	Neg A > 100 ft² psf

Lower Surface	30.11-1A/B	16.67	-16.67	16.00	-16.00	16.00	-16.00	16.00	-16.00
Upper Surface	30.11-1A/B	16.67	-23.97	16.00	-21.46	16.00	-18.14	16.00	-16.00
Upr & Lwr Surf	30.11-1A/B	18.76	-16.00	17.19	-16.00	16.00	-16.00	16.00	-16.00

** A is effective wind area for C&C: Span Length * Effective Width*
** Effective width need not be less than 1/3 of the span length*
** Maximum and minimum values of pressure shown.*
** + Pressures acting toward surface, - Pressures acting away from surface*
** Per § 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}*

Steel Beam

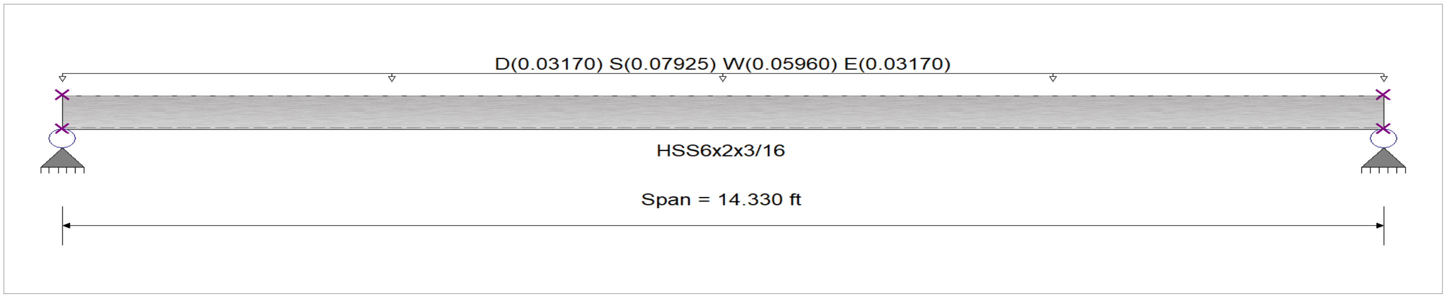
DESCRIPTION: Typical Canopy Beam Design (Wind Uplift Controls)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 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 calculated and added to loading
Uniform Load : D = 0.010, S = 0.0250, W = 0.01880, E = 0.010 ksf, Tributary Width = 3.170 ft, (Typical (Wind Uplift))

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.308** : 1	Maximum Shear Stress Ratio =	**0.029** : 1
Section used for this span	**HSS6x2x3/16**	Section used for this span	**HSS6x2x3/16**
Mu : Applied	5.286 k-ft	Vu : Applied	1.476 k
Mn * Phi : Allowable	17.175 k-ft	Vn * Phi : Allowable	51.471 k
Load Combination	+1.20D+1.60S+0.50W	Load Combination	+1.20D+1.60S+0.50W
		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	0.248 in	Ratio =	693	>=360	Span: 1 : S Only
Max Upward Transient Deflection	-0.187 in	Ratio =	921	>=360	Span: 1 : -W
Max Downward Total Deflection	0.399 in	Ratio =	431	>=180	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	-0.035 in	Ratio =	4956	>=180	Span: 1 : +0.60D-0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 14.33 ft	1	0.086	0.008	1.48		1.48	19.08	17.18	1.14	1.00	0.41	57.19	51.47
+1.20D													
Dsgn. L = 14.33 ft	1	0.074	0.007	1.27		1.27	19.08	17.18	1.14	1.00	0.35	57.19	51.47
+1.20D+0.50S													
Dsgn. L = 14.33 ft	1	0.133	0.012	2.28		2.28	19.08	17.18	1.14	1.00	0.64	57.19	51.47
+1.20D+0.50W													
Dsgn. L = 14.33 ft	1	0.118	0.011	2.03		2.03	19.08	17.18	1.14	1.00	0.57	57.19	51.47
+1.20D-0.50W													
Dsgn. L = 14.33 ft	1	0.029	0.003	0.50		0.50	19.08	17.18	1.14	1.00	0.14	57.19	51.47
+1.20D+1.60S													
Dsgn. L = 14.33 ft	1	0.263	0.025	4.52		4.52	19.08	17.18	1.14	1.00	1.26	57.19	51.47
+1.20D+1.60S+0.50W													
Dsgn. L = 14.33 ft	1	0.308	0.029	5.29		5.29	19.08	17.18	1.14	1.00	1.48	57.19	51.47
+1.20D+1.60S-0.50W													
Dsgn. L = 14.33 ft	1	0.219	0.020	3.76		3.76	19.08	17.18	1.14	1.00	1.05	57.19	51.47
+1.20D+W													
Dsgn. L = 14.33 ft	1	0.163	0.015	2.80		2.80	19.08	17.18	1.14	1.00	0.78	57.19	51.47
+1.20D-W													
Dsgn. L = 14.33 ft	1	0.015	0.001		-0.26	0.26	19.08	17.18	1.14	1.00	0.07	57.19	51.47
+1.20D+0.50S+W													
Dsgn. L = 14.33 ft	1	0.222	0.021	3.81		3.81	19.08	17.18	1.14	1.00	1.06	57.19	51.47

Steel Beam

Project File: gensler_allister.ec6

LIC# : KW-06018139, Build:20.23.08.01 KPFF CONSULTING ENGINEERS SEA (c) ENERCALC INC 1983-2023

DESCRIPTION: Typical Canopy Beam Design (Wind Uplift Controls)

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.20D+0.50S-W													
Dsgn. L = 14.33 ft	1	0.044	0.004	0.75		0.75	19.08	17.18	1.14	1.00	0.21	57.19	51.47
+0.90D+W													
Dsgn. L = 14.33 ft	1	0.144	0.013	2.48		2.48	19.08	17.18	1.14	1.00	0.69	57.19	51.47
+0.90D-W													
Dsgn. L = 14.33 ft	1	0.034	0.003		-0.58	0.58	19.08	17.18	1.14	1.00	0.16	57.19	51.47
+1.20D+0.20S+E													
Dsgn. L = 14.33 ft	1	0.145	0.013	2.49		2.49	19.08	17.18	1.14	1.00	0.69	57.19	51.47
+0.90D+E													
Dsgn. L = 14.33 ft	1	0.103	0.010	1.76		1.76	19.08	17.18	1.14	1.00	0.49	57.19	51.47

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.3987	7.206		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.1287	7.206		0.0000	0.000
+D+S	1	0.3768	7.206		0.0000	0.000
+D+0.750S	1	0.3148	7.206		0.0000	0.000
+D+0.60W	1	0.2406	7.206		0.0000	0.000
+D-0.60W	1	0.0168	7.206		0.0000	0.000
+D+0.450W	1	0.2127	7.206		0.0000	0.000
+D-0.450W	1	0.0448	7.206		0.0000	0.000
+D+0.750S+0.450W	1	0.3987	7.206		0.0000	0.000
+D+0.750S-0.450W	1	0.2308	7.206		0.0000	0.000
+0.60D+0.60W	1	0.1892	7.206		0.0000	0.000
+D+0.70E	1	0.1982	7.206		0.0000	0.000
+D-0.70E	1	0.0593	7.206		0.0000	0.000
+D+0.750S+0.5250E	1	0.3668	7.206		0.0000	0.000
+D+0.750S-0.5250E	1	0.2627	7.206		0.0000	0.000
+0.60D+0.70E	1	0.1467	7.206		0.0000	0.000
+0.60D-0.70E	1	0.0078	7.206		0.0000	0.000
S Only	1	0.2481	7.206		0.0000	0.000
W Only	1	0.1865	7.206		0.0000	0.000
E Only	1	0.0992	7.206		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

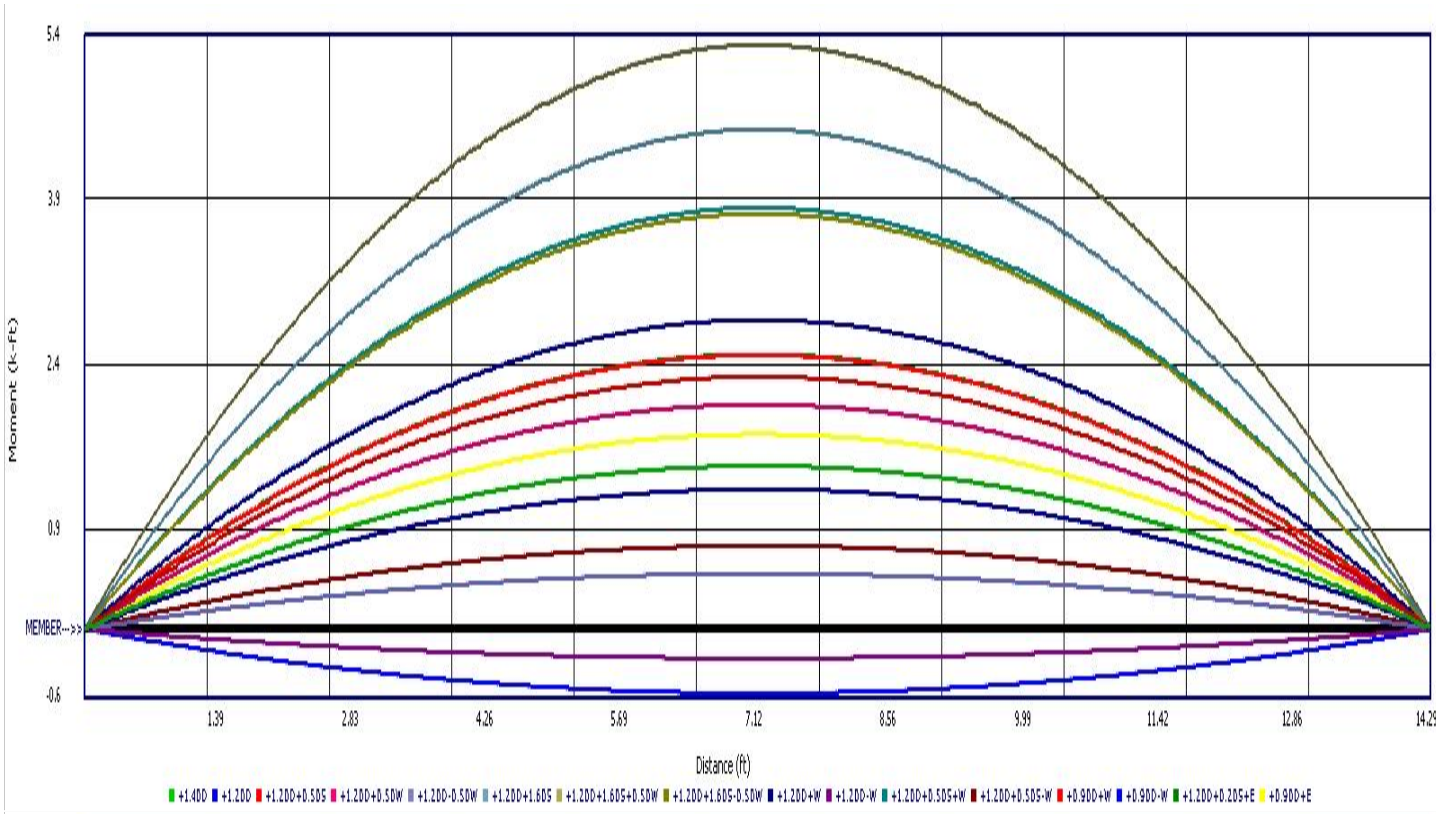
Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.913	0.913
Max Upward from Load Combinations	0.913	0.913
Max Upward from Load Cases	0.568	0.568
Max Downward from all Load Conditions (Resis	-0.427	-0.427
Max Downward from Load Combinations (Resi:	-0.427	-0.427
D Only	0.295	0.295
+D+S	0.862	0.862
+D+0.750S	0.720	0.720
+D+0.60W	0.551	0.551
+D-0.60W	0.038	0.038
+D+0.450W	0.487	0.487
+D-0.450W	0.102	0.102
+D+0.750S+0.450W	0.913	0.913
+D+0.750S-0.450W	0.528	0.528
+0.60D+0.60W	0.433	0.433
+0.60D-0.60W	-0.079	-0.079
+D+0.70E	0.454	0.454
+D+0.750S+0.5250E	0.840	0.840
+0.60D+0.70E	0.336	0.336
S Only	0.568	0.568
W Only	0.427	0.427
-W	-0.427	-0.427
E Only	0.227	0.227

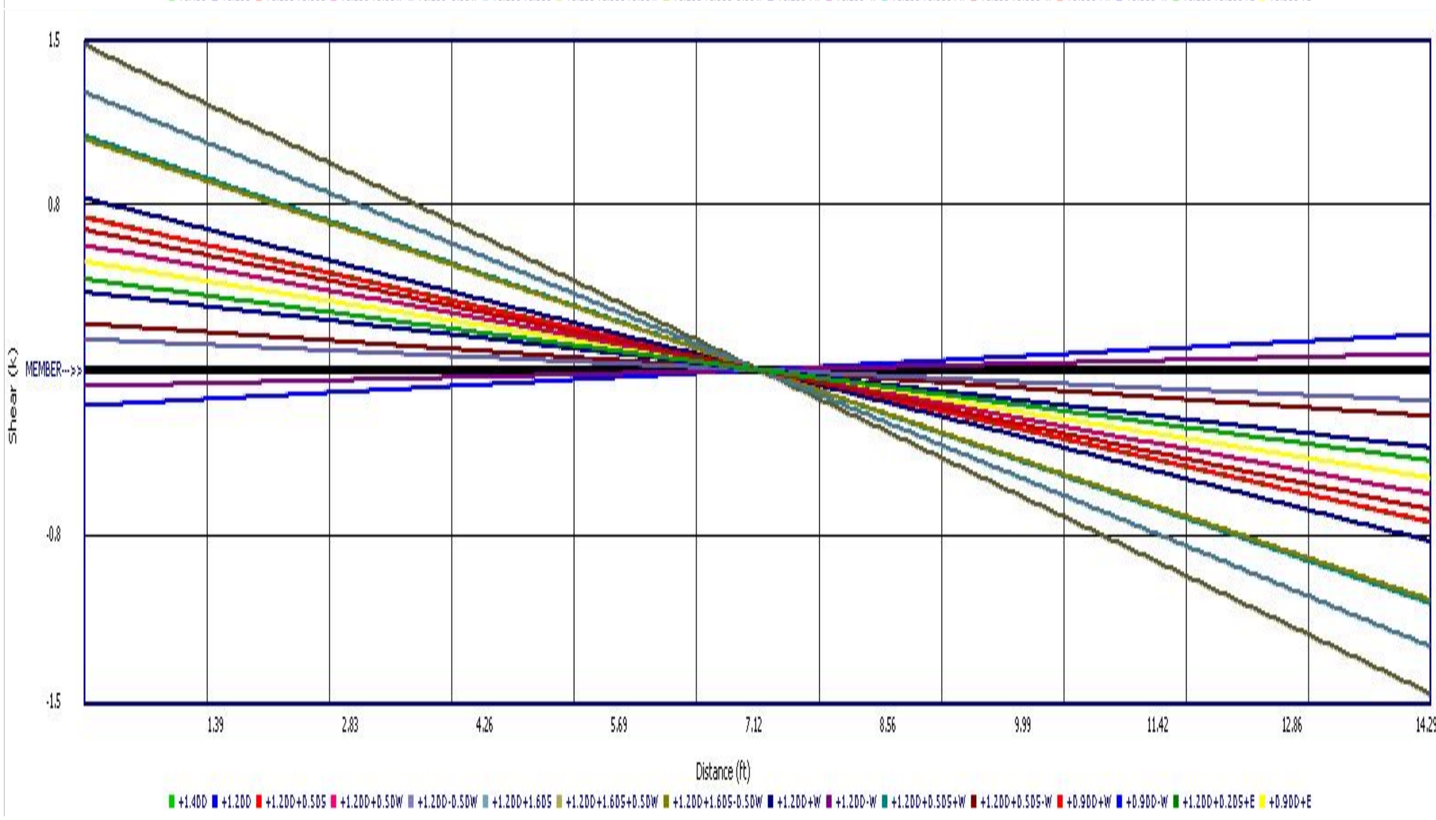
Steel Beam

Project File: gensler_allister.ec6

LIC# : KW-06018139, Build:20.23.08.01 KPFF CONSULTING ENGINEERS SEA (c) ENERCALC INC 1983-2023

DESCRIPTION: Typical Canopy Beam Design (Wind Uplift Controls)





Steel Beam

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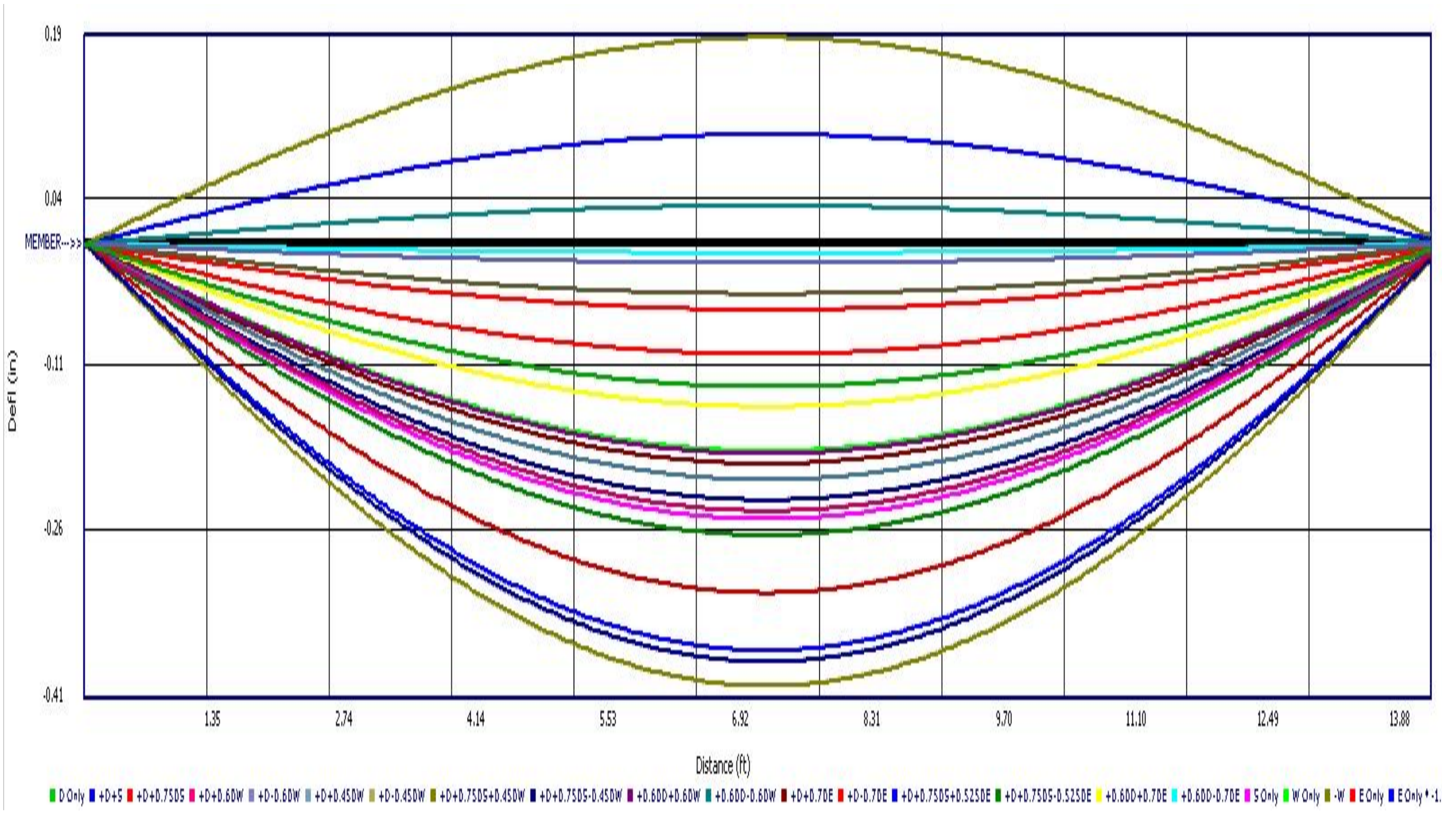
DESCRIPTION: Typical Canopy Beam Design (Wind Uplift Controls)



Steel Beam

Project File: gensler_allister.ec6

LIC# : KW-06018139, Build:20.23.08.01   KPFF CONSULTING ENGINEERS   SEA

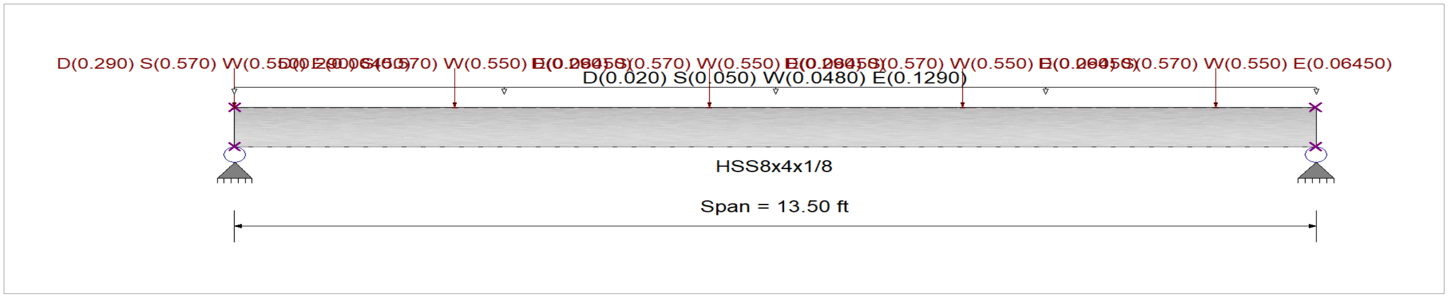
DESCRIPTION: HSS Headers at Pergola

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 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 calculated and added to loading
Load(s) for Span Number 1

Point Load : D = 0.290, S = 0.570, W = 0.550, E = 0.06450 k @ 0.0 ft, (Canopy Beam 1)

Point Load : D = 0.290, S = 0.570, W = 0.550, E = 0.06450 k @ 2.750 ft, (Canopy Beam 2)

Point Load : D = 0.290, S = 0.570, W = 0.550, E = 0.06450 k @ 5.930 ft, (Canopy Beam 3)

Point Load : D = 0.290, S = 0.570, W = 0.550, E = 0.06450 k @ 9.083 ft, (Canopy Beam 4)

Point Load : D = 0.290, S = 0.570, W = 0.550, E = 0.06450 k @ 12.250 ft, (Canopy Beam 5)

Uniform Load : D = 0.010, S = 0.0250, W = 0.0240, E = 0.06450 ksf, Tributary Width = 2.0 ft, (Typical Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.625** : 1	Maximum Shear Stress Ratio =	**0.101** : 1
Section used for this span	**HSS8x4x1/8**	Section used for this span	**HSS8x4x1/8**
Mu : Applied	14.428 k-ft	Vu : Applied	4.356 k
Mn * Phi : Allowable	23.099 k-ft	Vn * Phi : Allowable	43.043 k
Load Combination	+1.20D+1.60S+0.50W	Load Combination	+1.20D+1.60S+0.50W
		Location of maximum on span	13.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.260 in	Ratio =	621	>=360	Span: 1 : S Only
Max Upward Transient Deflection	-0.251 in	Ratio =	644	>=360	Span: 1 : -W
Max Downward Total Deflection	0.446 in	Ratio =	363	>=180	Span: 1 : +D+0.750S+0.450W
Max Upward Total Deflection	-0.068 in	Ratio =	2377	>=180	Span: 1 : +0.60D-0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 13.50 ft	1	0.170	0.028	3.92		3.92	25.67	23.10	1.15	1.00	1.18	47.83	43.04
+1.20D													
Dsgn. L = 13.50 ft	1	0.146	0.024	3.36		3.36	25.67	23.10	1.15	1.00	1.02	47.83	43.04
+1.20D+0.50S													
Dsgn. L = 13.50 ft	1	0.261	0.042	6.02		6.02	25.67	23.10	1.15	1.00	1.82	47.83	43.04
+1.20D+0.50W													
Dsgn. L = 13.50 ft	1	0.256	0.042	5.92		5.92	25.67	23.10	1.15	1.00	1.79	47.83	43.04

Steel Beam

DESCRIPTION: HSS Headers at Pergola

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.20D-0.50W													
Dsgn. L = 13.50 ft	1	0.035	0.006	0.80		0.80	25.67	23.10	1.15	1.00	0.24	47.83	43.04
+1.20D+1.60S													
Dsgn. L = 13.50 ft	1	0.514	0.083	11.87		11.87	25.67	23.10	1.15	1.00	3.58	47.83	43.04
+1.20D+1.60S+0.50W													
Dsgn. L = 13.50 ft	1	0.625	0.101	14.43		14.43	25.67	23.10	1.15	1.00	4.36	47.83	43.04
+1.20D+1.60S-0.50W													
Dsgn. L = 13.50 ft	1	0.403	0.065	9.31		9.31	25.67	23.10	1.15	1.00	2.81	47.83	43.04
+1.20D+W													
Dsgn. L = 13.50 ft	1	0.367	0.060	8.49		8.49	25.67	23.10	1.15	1.00	2.56	47.83	43.04
+1.20D-W													
Dsgn. L = 13.50 ft	1	0.076	0.012		-1.76	1.76	25.67	23.10	1.15	1.00	0.53	47.83	43.04
+1.20D+0.50S+W													
Dsgn. L = 13.50 ft	1	0.482	0.078	11.14		11.14	25.67	23.10	1.15	1.00	3.36	47.83	43.04
+1.20D+0.50S-W													
Dsgn. L = 13.50 ft	1	0.039	0.006	0.90		0.90	25.67	23.10	1.15	1.00	0.27	47.83	43.04
+0.90D+W													
Dsgn. L = 13.50 ft	1	0.331	0.054	7.65		7.65	25.67	23.10	1.15	1.00	2.31	47.83	43.04
+0.90D-W													
Dsgn. L = 13.50 ft	1	0.113	0.018		-2.60	2.60	25.67	23.10	1.15	1.00	0.79	47.83	43.04
+1.20D+0.20S+E													
Dsgn. L = 13.50 ft	1	0.338	0.055	7.80		7.80	25.67	23.10	1.14	1.00	2.35	47.83	43.04
+0.90D+E													
Dsgn. L = 13.50 ft	1	0.256	0.041	5.90		5.90	25.67	23.10	1.14	1.00	1.78	47.83	43.04

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.4460	6.789		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.1376	6.789		0.0000	0.000
+D+S	1	0.3982	6.789		0.0000	0.000
+D+0.750S	1	0.3330	6.789		0.0000	0.000
+D+0.60W	1	0.2883	6.789		0.0000	0.000
+D+0.450W	1	0.2506	6.789		0.0000	0.000
+D-0.450W	1	0.0245	6.789		0.0000	0.000
+D+0.750S+0.450W	1	0.4460	6.789		0.0000	0.000
+D+0.750S-0.450W	1	0.2200	6.789		0.0000	0.000
+0.60D+0.60W	1	0.2332	6.789		0.0000	0.000
+D+0.70E	1	0.2558	6.789		0.0000	0.000
+D+0.750S+0.5250E	1	0.4217	6.789		0.0000	0.000
+0.60D+0.70E	1	0.2008	6.789		0.0000	0.000
S Only	1	0.2606	6.789		0.0000	0.000
W Only	1	0.2512	6.789		0.0000	0.000
E Only	1	0.1689	6.789		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	3.280	2.746
Max Upward from Load Combinations	3.280	2.746
Max Upward from Load Cases	1.920	1.605
Max Downward from all Load Conditions (Resis	-1.851	-1.547
Max Downward from Load Combinations (Resi	-1.851	-1.547
D Only	1.007	0.846
+D+S	2.927	2.451
+D+0.750S	2.447	2.050
+D+0.60W	2.118	1.774
+D-0.60W	-0.104	-0.082
+D+0.450W	1.840	1.542
+D-0.450W	0.174	0.150
+D+0.750S+0.450W	3.280	2.746
+D+0.750S-0.450W	1.614	1.354

Steel Beam

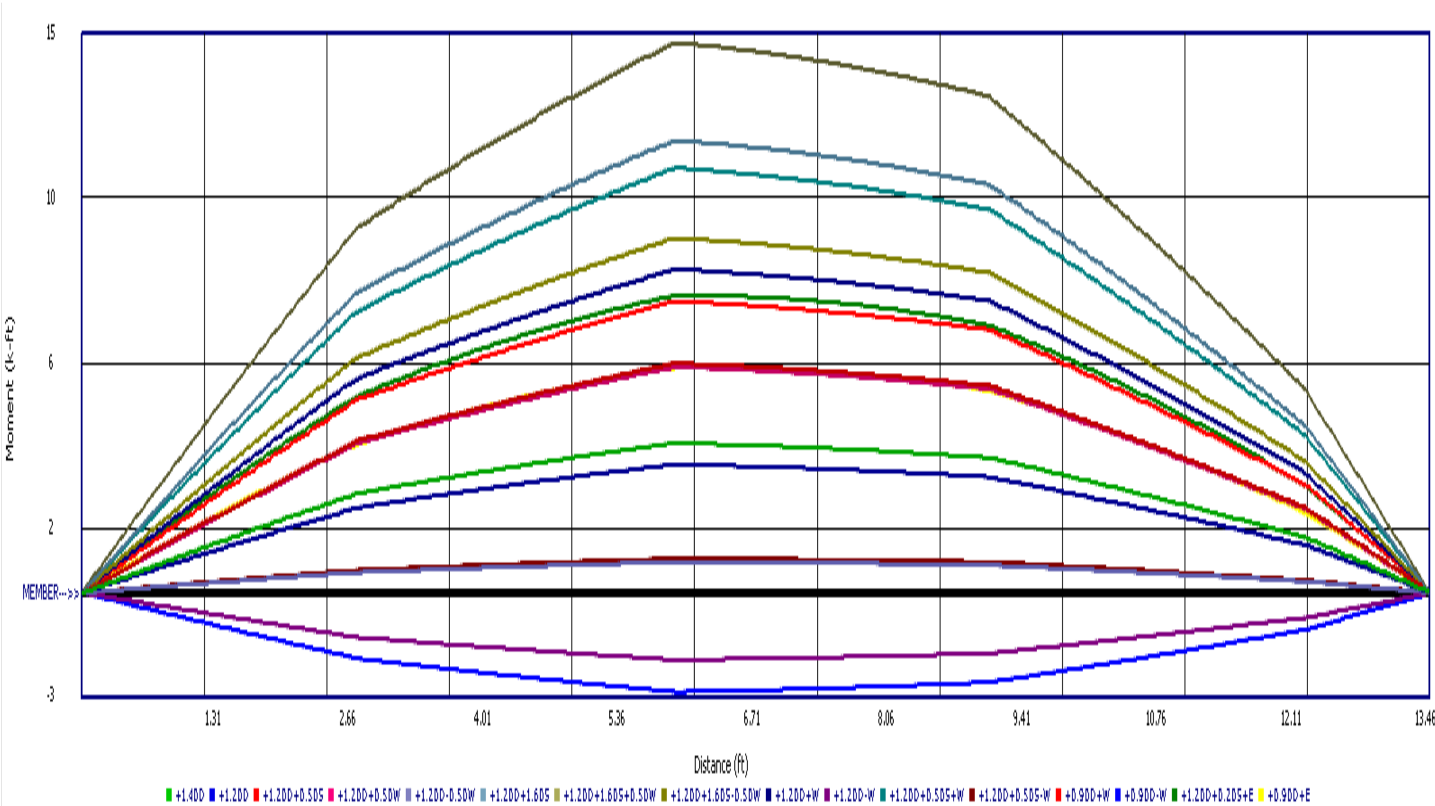
Project File: gensler_allister.ec6

DESCRIPTION: HSS Headers at Pergola

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+0.60D+0.60W	1.715	1.436
+0.60D-0.60W	-0.507	-0.420
+D+0.70E	1.742	1.556
+D+0.750S+0.5250E	2.998	2.582
+0.60D+0.70E	1.339	1.218
S Only	1.920	1.605
W Only	1.851	1.547
-W	-1.851	-1.547
E Only	1.050	1.014

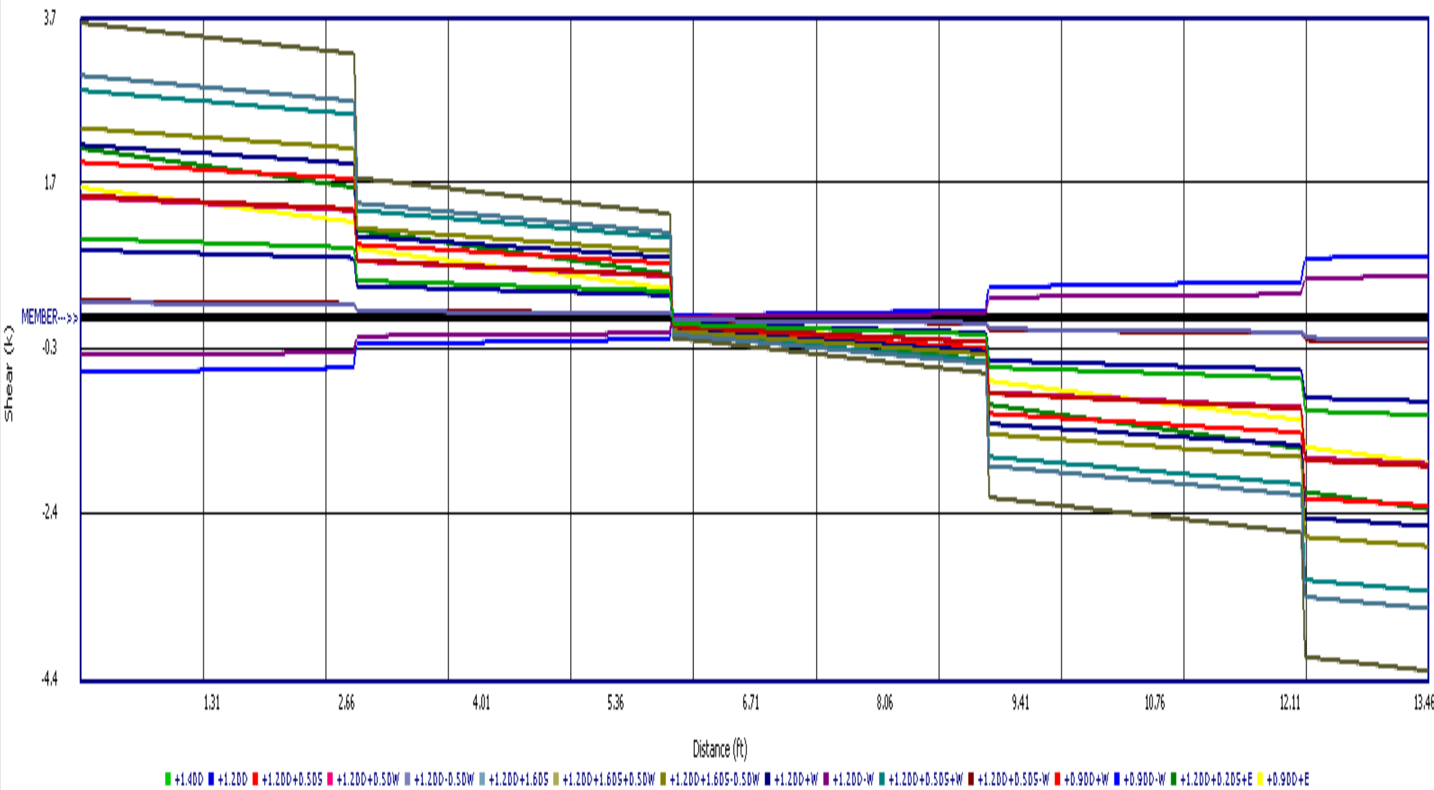


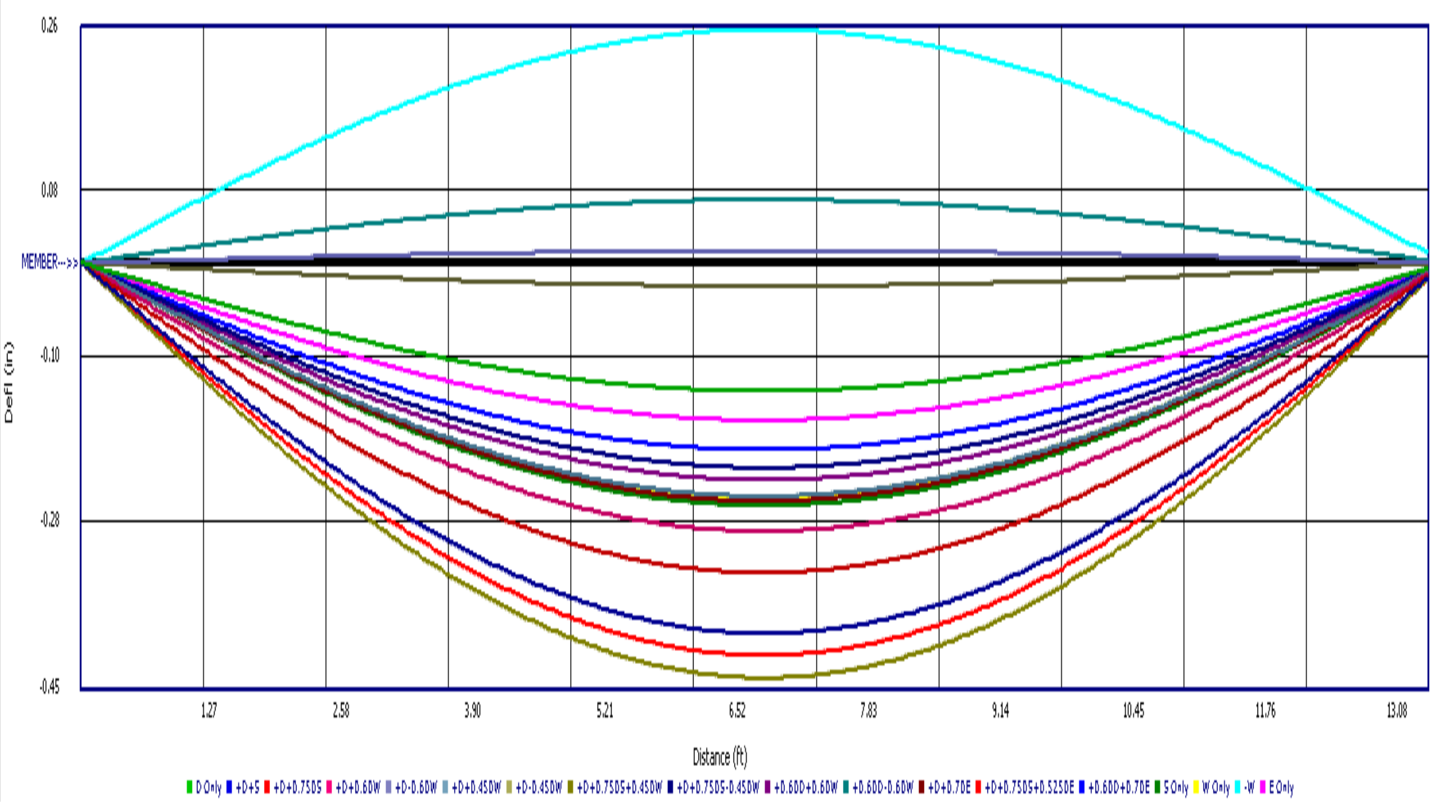
Steel Beam

Project File: gensler_allister.ec6

LIC# : KW-06018139, Build:20.23.08.01 KPFF CONSULTING ENGINEERS SEA (c) ENERCALC INC 1983-2023

DESCRIPTION: HSS Headers at Pergola





Steel Column

Project File: gensler_allister.ec6

LIC# : KW-06018139, Build:20.23.08.01 KPFF CONSULTING ENGINEERS SEA (c) ENERCALC INC 1983-2023

DESCRIPTION: Canopy Support Posts (Typ, Uplift Wind Controlled)

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS8x4x1/8**

Analysis Method :	Load Resistance Factor
Steel Stress Grade	, A500, Grade C, Fy = 50 ksi, Carbon Steel
Fy : Steel Yield	50.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	12.0 ft
Top & Bottom Fixity	Top & Bottom Pinned
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K = 1.0

Applied Loads

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

Column self weight included : 118.320 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.0 ft, D = 1.010, S = 1.920, W = 1.850, E = 1.050 k
BENDING LOADS . . .
Seismics Horizontal Load: Lat. Point Load at 12.0 ft creating Mx-x, E = 0.630 k
Seismics Horizontal Load: Lat. Point Load at 12.0 ft creating My-y, E = 0.630 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.07397**	: 1
Load Combination	+1.20D+1.60S+0.50W	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pu	5.351	k
0.9 * Pn	72.342	k
Mu-x	0.0	k-ft
0.9 * Mn-x :	23.099	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	15.325	k-ft
PASS Maximum Shear Stress Rati	**0.03305**	: 1
Load Combination	+1.20D+0.20S+E	
Location of max.above base	12.0	ft
At maximum location values are . . .		
Vu : Applied	0.630	k
Vn * Phi : Allowable	19.063	k

Maximum Load Reactions . .

Top along X-X	0.630 k
Bottom along X-X	0.0 k
Top along Y-Y	0.630 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 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
+1.40D	0.022	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D	0.019	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+0.50S	0.032	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+0.50W	0.032	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D-0.50W	0.006	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+1.60S	0.061	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+1.60S+0.50W	0.074	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+1.60S-0.50W	0.048	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+W	0.044	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D-W	0.000	PASS	12.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+0.50S+W	0.058	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+0.50S-W	0.006	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+0.90D+W	0.040	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+0.90D-W	0.000	PASS	12.00 ft	1.00	1.00	49.32	84.21	0.000	PASS	0.00 ft
+1.20D+0.20S+E	0.039	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.033	PASS	12.00 ft
+0.90D+E	0.029	PASS	0.00 ft	1.00	1.00	49.32	84.21	0.033	PASS	12.00 ft

Steel Column

DESCRIPTION: Canopy Support Posts (Typ, Uplift Wind Controlled)

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	1.128								
+D+S	3.048								
+D+0.750S	2.568								
+D+0.60W	2.238								
+D-0.60W	0.018								
+D+0.450W	1.961								
+D-0.450W	0.296								
+D+0.750S+0.450W	3.401								
+D+0.750S-0.450W	1.736								
+0.60D+0.60W	1.787								
+0.60D-0.60W	-0.433								
+D+0.70E	1.863		0.441		0.441				
+D+0.750S+0.5250E	3.120		0.331		0.331				
+0.60D+0.70E	1.412		0.441		0.441				
S Only	1.920								
W Only	1.850								
-W	-1.850								
E Only	1.050		0.630		0.630				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	3.401								
"	Minimum	-1.850								
Reaction, X-X Axis Base	Maximum	1.128								
"	Minimum	1.128								
Reaction, Y-Y Axis Base	Maximum	1.128								
"	Minimum	1.128								
Reaction, X-X Axis Top	Maximum	1.050		0.630		0.630				
"	Minimum	1.128								
Reaction, Y-Y Axis Top	Maximum	1.128								
"	Minimum	1.128								
Moment, X-X Axis Base	Maximum	1.128								
"	Minimum	1.128								
Moment, Y-Y Axis Base	Maximum	1.128								
"	Minimum	1.128								
Moment, X-X Axis Top	Maximum	1.128								
"	Minimum	1.128								
Moment, Y-Y Axis Top	Maximum	1.128								
"	Minimum	1.128								

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
+D+0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D-0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D-0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S-0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D-0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Column

DESCRIPTION: Canopy Support Posts (Typ, Uplift Wind Controlled)

Maximum Deflections for Load Combinations

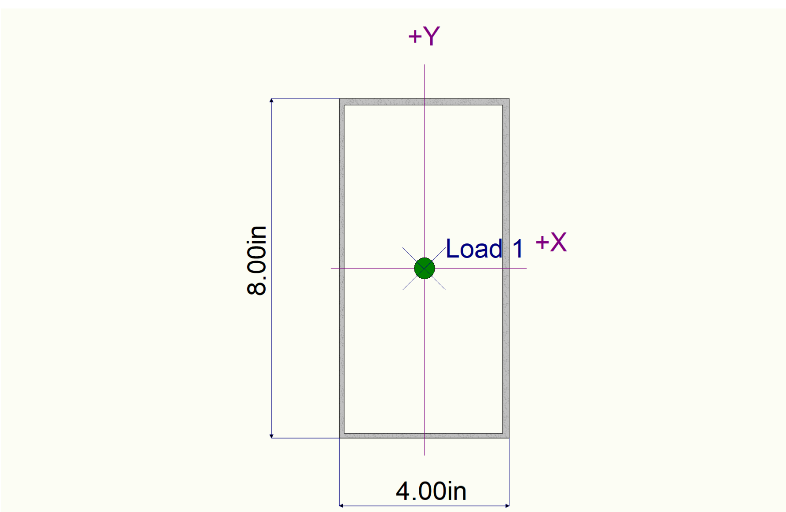
Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
-W	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

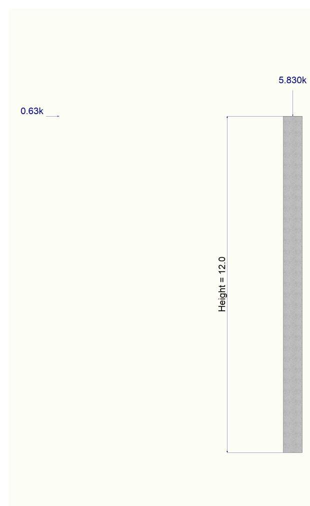
Steel Section Properties : HSS8x4x1/8

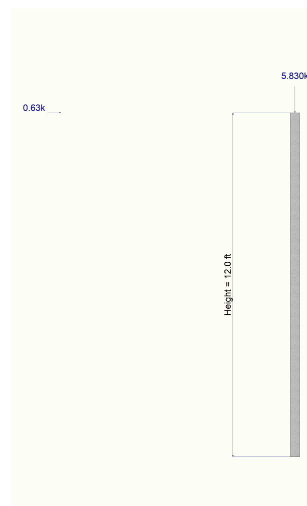
Depth	=	8.000 in	I xx	=	22.90 in^4	J	=	18.700 in^4
Design Thick	=	0.116 in	S xx	=	5.73 in^3	Cw	=	7.10 in^6
Width	=	4.000 in	R xx	=	2.920 in			
Wall Thick	=	0.125 in	Zx	=	7.020 in^3			
Area	=	2.700 in^2	I yy	=	7.900 in^4	C	=	7.100 in^3
Weight	=	9.860 plf	S yy	=	3.950 in^3			
			R yy	=	1.710 in			
			Zy	=	4.360 in^3			

Ycg = 0.000 in

Sketches







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Company:	KPFF Consulting Engineers	Page:	1
Address:	1601 Fifth Avenue, Ste. 1600, Seattle WA, 98101	Specifier:	Luke Small (LJS)
Phone I Fax:	206-622-5822 |	E-Mail:	luke.small@kpff.com
Design:	Canopy Beam Post-Installed Plate	Date:	11/17/2023
Fastening point:	Existing Building		

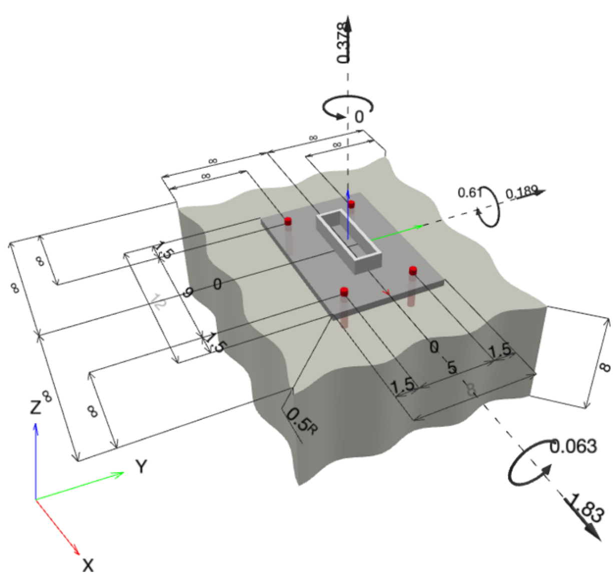
Specifier's comments: Scan for and avoid existing reabr in existing concrete wall prior to installation.

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (3)**
Item number:	418072 KH-EZ 1/2"x3 1/2"
Effective embedment depth:	$h_{ef,act}$ = 2.160 in., h_{nom} = 3.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS6X2X.250; (L x W x T) = 6.000 in. x 2.000 in. x 0.250 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (a))

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

Geometry [in.] & Loading [kip, ft.kip]



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Company:	KPFF Consulting Engineers	Page:	2
Address:	1601 Fifth Avenue, Ste. 1600, Seattle WA, 98101	Specifier:	Luke Small (LJS)
Phone I Fax:	206-622-5822 |	E-Mail:	luke.small@kpff.com
Design:	Canopy Beam Post-Installed Plate	Date:	11/17/2023
Fastening point:	Existing Building		

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0.378; V_x = 1.830; V_y = 0.189; M_x = 0.06300; M_y = 0.61000; M_z = 0.00000;	yes	48

Company:	KPFF Consulting Engineers	Page:	3
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Design:	Canopy Beam Post-Installed Plate	Date:	11/17/2023
Fastening point:	Existing Building		

2 Proof I Utilization (Governing Cases)

		Design values [kip]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	Concrete Breakout Failure	0.959	2.024	48 / -	OK
Shear	Pryout Strength	1.840	8.466	- / 22	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.474	0.217	5/3	37	OK

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

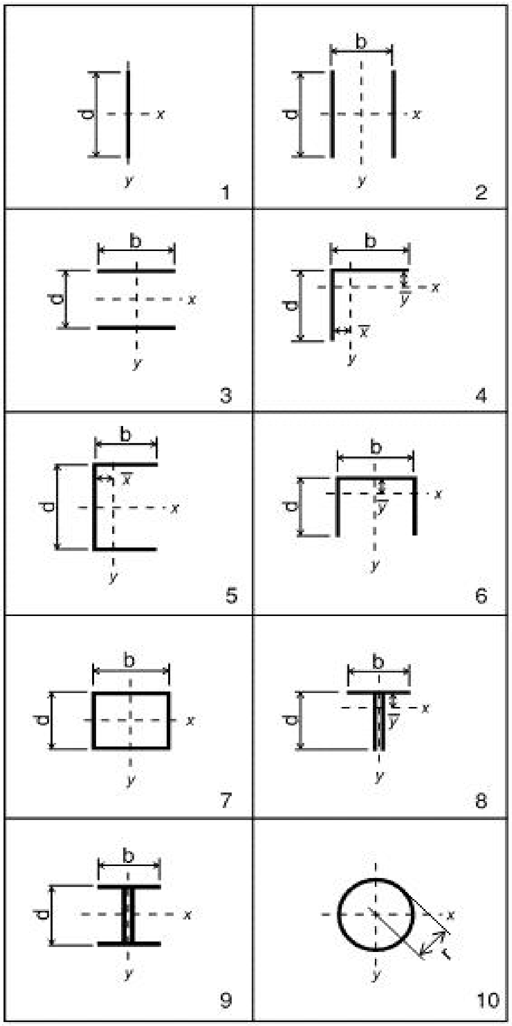
1601 5th Avenue, Suite 1600 Seattle, WA 98101 206 622-5822	project: Allister	by: LJS	sheet no.	
	location: Mercer Island, WA	date: 11/14/		
	client: Gensler		job no. 2300XXX	
	Canopy Post Base Plate			

Welded Connection Capacity (LRFD) - Elastic Method (Fillet Welds Treated as Lines)

For Axial, Direct Shear, Bending Moment, and Torsion

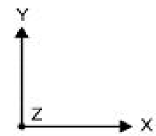
Connection ID: Conn Plate to HSS

WELD GEOMETRY TYPES



CONNECTION LOADS & JOINT INPUTS

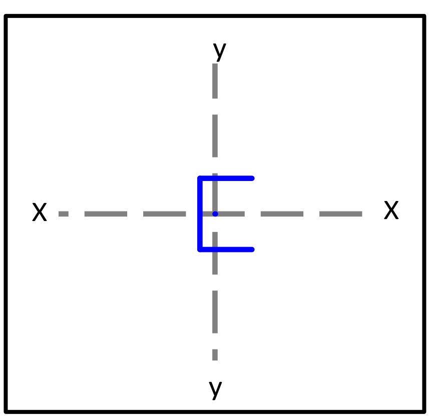
Axial, P_u =	0.2	kips	
Shear, $(V_u)_x$ =	0.4	kips	
Shear, $(V_u)_y$ =	1.8	kips	
Moment, $(M_u)_x$ =	0.0	kip-in	
Moment, $(M_u)_y$ =	0.8	kip-in	
Torque, $(T_u)_z$ =	7.3	kip-in	



Welded Joint Type:	5	*Weld Geometry Type*	
b, width =	2.75	in	
d, depth =	4.00	in	
r, radius =	5.0	in	
D =	3	*weld size in 1/16s of an inch*	
Area , A_w =	9.50	in	
Section Modulus, $S_{w,x}$ =	13.67	in^2	
Section Modulus, $S_{w,y}$ =	9.85	in^2	
Polar Moment of Inertia, J_w =	35.18	in^3	
Extreme Fiber x-dir, c_x =	1.95	in	
Extreme Fiber y-dir, c_y =	2.00	in	
$(P_u)/A_w$ =	0.02	kips/in	*Axial, z-direction*
$(V_u)_x/A_w$ =	0.04	kips/in	*Shear, x-direction*
$(V_u)_y/A_w$ =	0.19	kips/in	*Shear, y-direction*
$(M_u)_x/S_{w,x}$ =	0.00	kips/in	*Flexure, about x-axis*
$(M_u)_y/S_{w,y}$ =	0.08	kips/in	*Flexure, about y-axis*
$(T_u)_z c_y/J_w$ =	0.42	kips/in	*Torsion, about z-axis*
$(T_u)_z c_x/J_w$ =	0.41	kips/in	*Torsion, about z-axis*
Resultant Force, f_r =	0.76	kips/in	
Weld Capacity, ΦR_n =	4.18	kips/in	OK

$\Phi R_n = 0.75(0.6)(70\text{ ksi})(\sqrt{2}/2)(D)$

Thickness of Thinner Edge:	1/4	inches	
Min Weld Size =	1/8		OK
Max Weld Size =	3/16		OK



$$f_r = \sqrt{\left(\frac{V_{ux}}{A_w} + \frac{T_u c_y}{J_w}\right)^2 + \left(\frac{V_{uy}}{A_w} + \frac{T_u c_x}{J_w}\right)^2 + \left(\frac{P_u}{A_w} + \frac{M_{ux}}{S_{wx}} + \frac{M_{uy}}{S_{wy}}\right)^2}$$

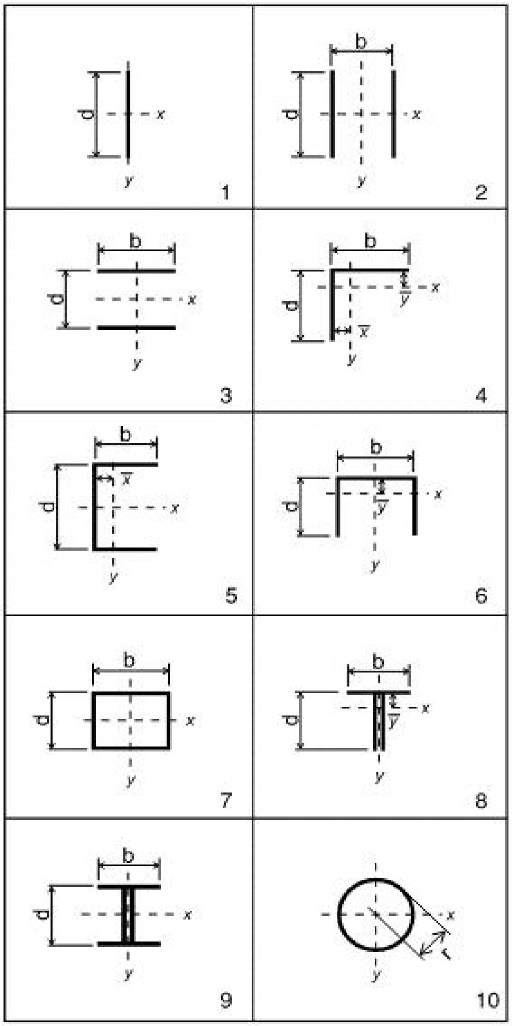
kpff 1601 5th Avenue, Suite 1600 Seattle, WA 98101 206 622-5822	project: Allister	by: LJS	sheet no.
	location: Mercer Island, WA	date: 11/14/	
	client: Gensler		job no. 2300XXX
	Canopy Post Base Plate		

Welded Connection Capacity (LRFD) - Elastic Method (Fillet Welds Treated as Lines)

For Axial, Direct Shear, Bending Moment, and Torsion

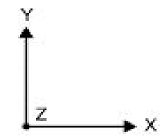
Connection ID: Conn Plate to HSS

WELD GEOMETRY TYPES



CONNECTION LOADS & JOINT INPUTS

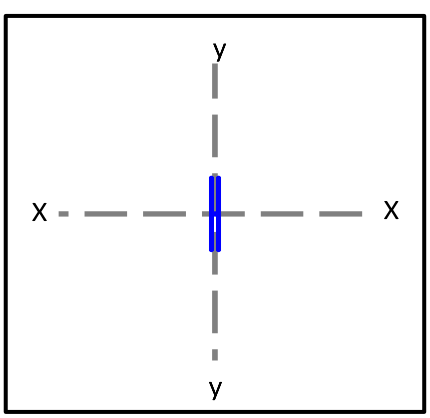
Axial, P_u =	0.4	kips	
Shear, $(V_u)_x$ =	0.2	kips	
Shear, $(V_u)_y$ =	1.8	kips	
Moment, $(M_u)_x$ =	7.3	kip-in	
Moment, $(M_u)_y$ =	0.8	kip-in	
Torque, $(T_u)_z$ =	0.0	kip-in	



Welded Joint Type:	2	*Weld Geometry Type*	
b, width =	0.38	in	
d, depth =	4.00	in	
r, radius =	5.0	in	
D =	3	*weld size in 1/16s of an inch*	
Area , A_w =	8.00	in	
Section Modulus, $S_{w,x}$ =	5.33	in^2	
Section Modulus, $S_{w,y}$ =	1.50	in^2	
Polar Moment of Inertia, J_w =	10.95	in^3	
Extreme Fiber x-dir, c_x =	0.19	in	
Extreme Fiber y-dir, c_y =	2.00	in	
$(P_u)/A_w$ =	0.05	kips/in	*Axial, z-direction*
$(V_u)_x/A_w$ =	0.02	kips/in	*Shear, x-direction*
$(V_u)_y/A_w$ =	0.23	kips/in	*Shear, y-direction*
$(M_u)_x/S_{w,x}$ =	1.37	kips/in	*Flexure, about x-axis*
$(M_u)_y/S_{w,y}$ =	0.50	kips/in	*Flexure, about y-axis*
$(T_u)_z c_y/J_w$ =	0.00	kips/in	*Torsion, about z-axis*
$(T_u)_z c_x/J_w$ =	0.00	kips/in	*Torsion, about z-axis*
Resultant Force, f_r =	1.94	kips/in	
Weld Capacity, ΦR_n =	4.18	kips/in	OK

$\Phi R_n = 0.75(0.6)(70\ ksi)(\sqrt{2}/2)(D)$

Thickness of Thinner Edge:	1/4	inches	
Min Weld Size =	1/8		OK
Max Weld Size =	3/16		OK



$$f_r = \sqrt{\left(\frac{V_{ux}}{A_w} + \frac{T_u c_y}{J_w}\right)^2 + \left(\frac{V_{uy}}{A_w} + \frac{T_u c_x}{J_w}\right)^2 + \left(\frac{P_u}{A_w} + \frac{M_{ux}}{S_{wx}} + \frac{M_{uy}}{S_{wy}}\right)^2}$$

Company:	KPFF Consulting Engineers	Page:	1
Address:	1601 Fifth Avenue, Ste. 1600, Seattle WA, 98101	Specifier:	Luke Small (LJS)
Phone I Fax:	206-622-5822 |	E-Mail:	luke.small@kpff.com
Design:	Post Base Plate	Date:	11/17/2023
Fastening point:	New Spread Footing		

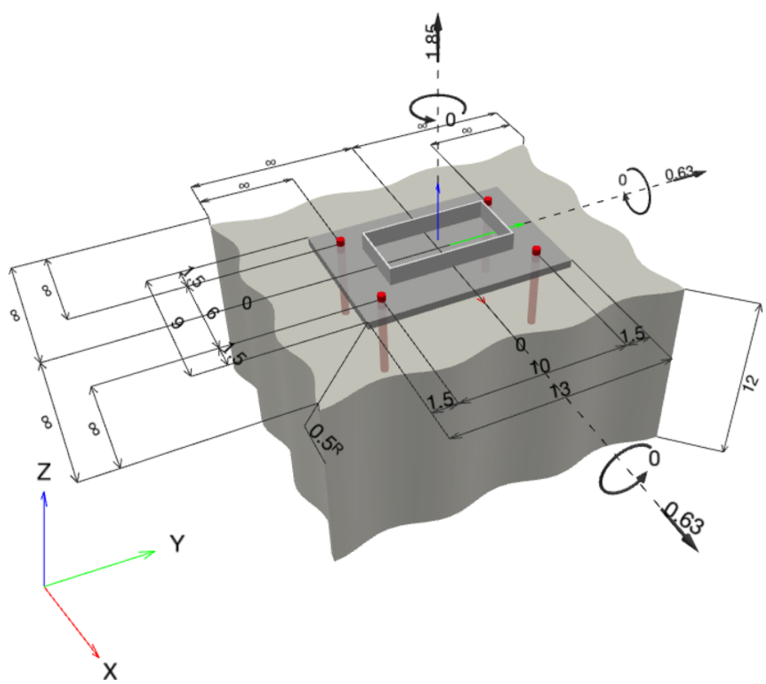
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1/2**
Item number:	not available
Effective embedment depth:	h_{ef} = 4.724 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 9.000 in. x 13.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS8X4X.125; (L x W x T) = 8.000 in. x 4.000 in. x 0.125 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar

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

Geometry [in.] & Loading [kip, ft.kip]



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Company:	KPFF Consulting Engineers	Page:	2
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Design:	Post Base Plate	Date:	11/17/2023
Fastening point:	New Spread Footing		

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1.850$; $V_x = 0.630$; $V_y = 0.630$; $M_x = 0.00000$; $M_y = 0.00000$; $M_z = 0.00000$;	no	8

Company:	KPFF Consulting Engineers	Page:	3
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Design:	Post Base Plate	Date:	11/17/2023
Fastening point:	New Spread Footing		

2 Proof I Utilization (Governing Cases)

		Design values [kip]		Utilization	
Loading	Proof	Load	Capacity	β_N / β_V [%]	Status
Tension	Pullout Strength	0.462	6.518	8 / -	OK
Shear	Steel Strength	0.223	3.212	- / 7	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.075	0.069	5/3	3	OK

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

project	Project Name	by	ENG	sheet no.
location	Project Location	date	X/XX/XX	
client	Client			job no.
	Subject			

1601 5th Avenue, Suite 1600
Seattle, WA 98101 206 622-5822

Welded Connection Capacity (LRFD) - Elastic Method (Fillet Welds Treated as Lines)

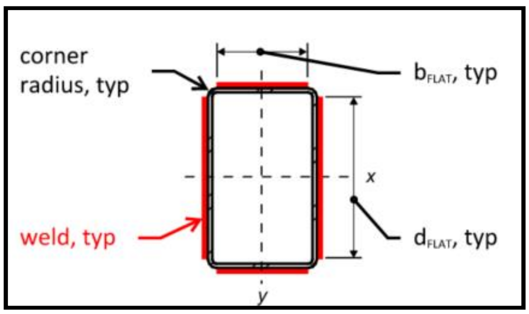
For Axial, Direct Shear, Bending Moment, and Torsion - Across HSS Flats

Connection ID: Post to Base Plate

HSS SECTION & GEOMETRY

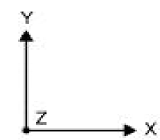
HSS Section HSS8X4X.250

b =	4.00	in
$h = d$ =	8.00	in
t_{nom} =	0.250	in, design thickness
$r = 2.25t_{nom}$ =	0.563	in, corner radius
=> b_{flat} =	2.88	in
=> d_{flat} =	6.88	in



CONNECTION LOADS & JOINT INPUTS

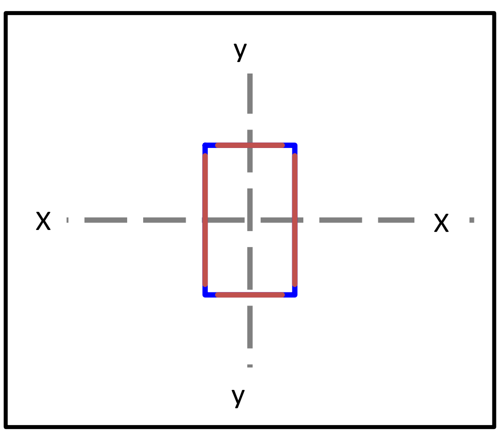
Axial, P_u =	1.9	kips
Shear, $(V_u)_x$ =	0.6	kips
Shear, $(V_u)_y$ =	0.6	kips
Moment, $(M_u)_x$ =	0.0	kip-in
Moment, $(M_u)_y$ =	0.0	kip-in
Torque, $(T_u)_z$ =	0.0	kip-in



Area , A_w =	19.50	in, $A_w = 2b_{flat} + 2d_{flat}$
Section Modulus, $S_{w,x}$ =	38.76	in^2, $S_{w,x} = d_{flat}^2/3 + b_{flat}d$
Section Modulus, $S_{w,y}$ =	30.26	in^2, $S_{w,y} = b_{flat}^2/3 + d_{flat}b$
Polar Moment of Inertia, J_w =	205.12	in^3, $J_w = d_{flat}(3b^2+d_{flat}^2)/6 + (b_{flat}^3+3b_{flat}d^2)/6$
Extreme Fiber x-dir, c_x =	2.00	in
Extreme Fiber y-dir, c_y =	4.00	in

$(P_u)/A_w$ =	0.09	kips/in	*Axial, z-direction*
$(V_u)_x/A_w$ =	0.03	kips/in	*Shear, x-direction*
$(V_u)_y/A_w$ =	0.03	kips/in	*Shear, y-direction*
$(M_u)_x/S_{w,x}$ =	0.00	kips/in	*Flexure, about x-axis*
$(M_u)_y/S_{w,y}$ =	0.00	kips/in	*Flexure, about y-axis*
$(T_u)_z c_y/J_w$ =	0.00	kips/in	*Torsion, about z-axis*
$(T_u)_z c_x/J_w$ =	0.00	kips/in	*Torsion, about z-axis*

HSS WELD PLOT



- Blue represents HSS Section
- Red represent welds at flats

Resultant Force, f_r =	0.11	kips/in	
D =	3	*weld size in 1/16s of an inch*	
Weld Capacity, ΦR_n =	4.18	kips/in	OK

$\Phi R_n = 0.75(0.6)(70\text{ ksi})(\sqrt{2}/2)(D)$

Thickness of Thinner Edge:	0.500	inches	
Min Weld Thickness:	3/16	in	OK
Max Weld Thickness:	7/16	in	OK

$$f_r = \sqrt{\left(\frac{V_{ux}}{A_w} + \frac{T_u c_y}{J_w}\right)^2 + \left(\frac{V_{uy}}{A_w} + \frac{T_u c_x}{J_w}\right)^2 + \left(\frac{P_u}{A_w} + \frac{M_{ux}}{S_{wx}} + \frac{M_{uy}}{S_{wy}}\right)^2}$$