

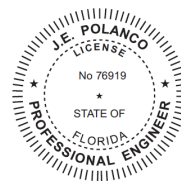
STRUCTURAL CALCULATIONS FOR:

ALUMINUM CANOPY

LOCATION: 7465 SW 127 ST, PINECREST, FL 33156

INDEX:

Cover	1
Calculation	2-43



Digitally signed
by Jose E Polanco
Date: 2025.07.17
09:12:21 -04'00'

Jose E. Polanco
P.E. No. 76919

MecaWind v2530

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Calculations Prepared by:

Date: Jun 13, 2025

General:

Reference Abbreviations: T: Table, F: Figure, E: Equation, \$: Section

Wind Load Standard	= ASCE 7-22	Basic Wind Speed	= 175.0 mph
Exposure Classification	= C	Risk Category	= II
Structure Type	= Canopy	Basis for Wind Pressures	= ASD
MWFRS Analysis Method	= None	C&C Analysis Method	= Ch 30 Pt 4
Dynamic Type of Structure	= Rigid	Advanced Options	= True
Base Elevation Of Structure	= 0.0000 ft	MWFRS Pressure Elevations	= Mean Ht
Topographic Effects	= None	Override Directionality Factor	= True
Manual - Directionality Factor	= 0.8500	Override Gust Factor	= True
Manual - Gust Factor	= 0.850	Override Minimum Pressure	= False

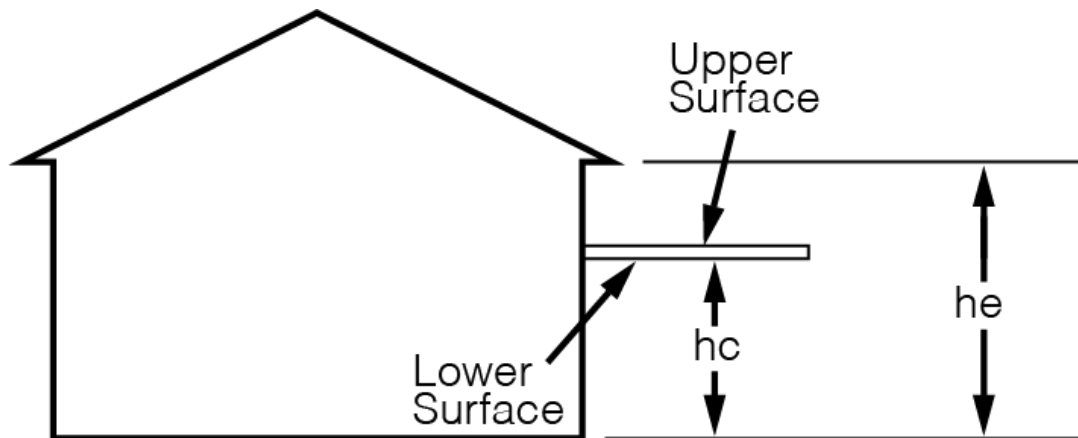
Building Inputs

h	= Mean Building Roof Height	= 12.0000 ft
he	= Mean Eave Height	= 9.5000 ft
hc	= Mean canopy height	= 8.0000 ft

Exposure Constants [T:26.11-1]:

α = 3-s Gust-speed exponent	= 9.800	Z_g = Nominal Ht of Boundary Layer	= 2460.000 ft
$\hat{\alpha}$ = Reciprocal of α	= 0.102	b = 3 sec gust speed factor	= 1.000
α_m = Mean hourly Wind-Speed Exponent	= 0.156	b_m = Mean hourly Windspeed Exponent	= 0.660
c = Turbulence Intensity Factor	= 0.200	ϵ = Integral Length Scale Exponent	= 0.2000

Components and Cladding (C&C) Wind Loads per Ch 30 Pt 4 Canopy



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

h = Mean structure height	= 12.000 ft	$K_h = 2.41 \cdot (15/Z_g)^{2/\alpha}$	= 0.851
K_{zt} = No Topographic Feature	= 1.000	K_d = Directionality (Manual)	= 0.85
LF = ASD Load Factor	= 0.60	K_e = Ground Elev Factor T:26.10-1	= 1.000
$q_h = .00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF_{E:26.10-1}$	= 40.04 psf		

C&C Wind Canopy Summary per Ch 30 Pt 4

All wind pressures include a Load Factor (LF) of 0.6

Zone	Reference	P_{max} A ≤ 10 ft ² psf	P_{min} A ≤ 10 ft ² psf	P_{max} A = 20 ft ² psf	P_{min} A = 20 ft ² psf	P_{max} A = 50 ft ² psf	P_{min} A = 50 ft ² psf	P_{max} A > 100 ft ² psf	P_{min} A > 100 ft ² psf
Lower Surface	F:30.9-1A/B	27.23	-27.23	25.18	-25.69	22.47	-23.66	20.42	-22.12
Upper Surface	F:30.9-1A/B	27.23	-39.14	25.18	-35.04	22.47	-29.62	20.42	-25.52
Upr & Lwr Surf	F:30.9-1A/B	30.63	-30.63	28.07	-28.07	24.68	-24.68	22.12	-22.12

P_{max}	= Maximum Pressure	P_{min}	= Minimum Pressure
Area	= Span • Width	Width	= Width of Component
Span	= Span of Component	Reference	= Applicable Reference from Standard
$\$30.2.2$	= C&C Min Pressure = 9.60 psf	Interpolate	= Interpolate for Areas between columns


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Product Approval

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FL #	FL7561-R7
Application Type	Revision
Code Version	2023
Application Status	Approved
Comments	
Archived	☐
Product Manufacturer	Elite Aluminum Corporation
Address/Phone/Email	4650 Lyons Technology Parkway Coconut Creek, FL 33073 (954) 949-3200 bpeacock@elitealuminum.com
Authorized Signature	Bruce Peacock bpeacock@elitealuminum.com
Technical Representative	Bruce Peacock
Address/Phone/Email	4650 Lyons Technology Parkway Coconut Creek, FL 33073 (954) 949-3200 bpeacock@elitealuminum.com
Quality Assurance Representative	
Address/Phone/Email	
Category	Roofing
Subcategory	Products Introduced as a Result of New Technology
Compliance Method	Evaluation Report from a Florida Registered Architect or a Licensed Florida Professional Engineer ☐ Evaluation Report - Hardcopy Received
Florida Engineer or Architect Name who developed the Evaluation Report	Do Kim, P.E.
Florida License	PE-49497
Quality Assurance Entity	QAI Laboratories
Quality Assurance Contract Expiration Date	12/31/2026
Validated By	James L. Buckner, P.E. @ CBUCK Engineering ☒ Validation Checklist - Hardcopy Received
Certificate of Independence	FL7561_R7_COI_certificate_of_independence.pdf
Referenced Standard and Year (of Standard)	
Equivalence of Product Standards Certified By	
Sections from the Code	1708.2

Product Approval Method

Method 2 Option B

Date Submitted

08/16/2023

Date Validated

08/21/2023

Date Pending FBC Approval

08/25/2023

Date Approved

10/17/2023

Summary of Products

FL #	Model, Number or Name	Description
7561.1	Aluminum/Aluminum Composite Panels	3"/4"/6"x0.024"x1lb EPS Composite Panel, 3"/4"/6"x0.032x1lb EPS Composite Panel, 3"/4"/6"x0.024"x2lb EPS Composite Panel, 3"/4"/6"x0.030"x2lb EPS Composite Panel,
Limits of Use Approved for use in HVHZ: Yes Approved for use outside HVHZ: Yes Impact Resistant: No Design Pressure: +80/-80 Other: In HVHZ, not to be used in structures considered living areas per FBC Section 1616 unless impact protection is provided. See installation drawing for nominal allowable design pressures and spans.		Installation Instructions FL7561_R7_II_2023 FBC-Elite Aluminum Corp Install Dwg.pdf Verified By: Do Kim, P.E. PE 49497 Created by Independent Third Party: Yes Evaluation Reports FL7561_R7_AE_FL 7561 Evaluation Report-2023 FBC.pdf Created by Independent Third Party: Yes

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Product Approval Accepts:

ELITE PANEL SPAN TABLES:

1. Net allowable loads are permitted to be multiplied by 1.67 to derive ultimate loads (psf).

ELITE ALUMINUM PANELS ARE LABELED WITH A FL7561 LABEL TO ENSURE BUILDING INSPECTOR THAT THE INSULATED PANELS INSTALLED ARE APPROVED FOR USE IN THE STATE OF FLORIDA.

**3" x 0.024 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	16.17	15.76	15.03	14.10
20	13.44	13.44	12.22	10.35
30	10.78	10.78	9.41	6.60
40	9.22	9.22	6.60	2.85
50	8.17	8.17	3.79	-
60	7.40	6.39	0.98	-
70	6.81	4.51	-	-
80	6.33	2.64	-	-

**4" x 0.024 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	19.00	19.00	17.17	16.53
20	15.01	15.01	15.01	13.95
30	12.50	12.50	12.50	11.38
40	10.97	10.97	10.97	8.80
50	9.92	9.92	9.44	6.22
60	9.13	9.13	7.51	3.64
70	8.52	8.52	5.58	1.07
80	8.02	8.02	3.64	-

**6" x 0.024 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	23.00	21.24	21.47	20.85
20	18.06	18.06	18.06	18.06
30	15.13	15.13	15.13	15.13
40	13.34	13.34	13.34	13.34
50	12.10	12.10	12.10	10.91
60	11.17	11.17	11.17	8.43
70	10.44	10.44	10.30	5.95
80	9.85	9.85	8.43	3.47

**3" x 0.032 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	17.50	17.50	16.91	15.96
20	16.64	15.96	14.06	12.16
30	15.17	14.06	11.21	8.36
40	13.69	12.16	8.36	4.56
50	12.22	10.26	5.51	0.76
60	10.75	8.36	2.66	-
70	9.27	6.46	-	-
80	7.80	4.56	-	-

**4" x 0.032 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	20.50	20.50	20.11	19.24
20	19.61	19.24	17.49	15.74
30	18.17	17.49	14.87	12.24
40	16.72	15.74	12.24	8.74
50	15.28	13.99	9.62	5.25
60	13.84	12.24	7.00	1.75
70	12.40	10.49	4.38	-
80	10.95	8.74	1.75	-

**6" x 0.032 x 1 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	24.00	24.00	24.00	23.42
20	23.34	23.21	21.82	20.22
30	22.10	21.63	19.42	17.02
40	20.86	20.05	17.02	13.82
50	19.62	18.47	14.62	10.62
60	18.38	16.89	12.22	7.42
70	17.14	15.30	9.82	4.22
80	15.91	13.72	7.42	1.02

**3" x 0.024 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	19.33	18.95	18.31	17.66
20	18.11	17.66	16.36	15.06
30	16.80	16.36	14.41	12.46
40	15.49	15.06	12.46	9.86
50	14.18	13.76	10.51	7.26
60	12.87	12.46	8.57	4.67
70	11.57	11.16	6.62	2.07
80	10.26	9.86	4.67	-

**4" x 0.024 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	21.97	21.97	21.52	20.97
20	20.77	20.77	19.86	18.76
30	19.57	19.57	18.21	16.55
40	18.36	18.36	16.55	14.34
50	17.16	17.16	14.89	12.13
60	15.96	15.96	13.24	9.93
70	14.75	14.75	11.58	7.72
80	13.55	13.55	9.93	5.51

**6" x 0.024 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	23.93	23.93	23.98	23.60
20	23.20	23.20	23.03	22.46
30	22.47	22.47	22.18	21.33
40	21.75	21.75	21.33	20.20
50	21.02	21.02	20.49	19.07
60	20.29	20.29	19.64	17.94
70	19.57	19.57	18.79	16.81
80	18.84	18.84	17.94	15.68

**3" x 0.030 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	20.11	20.03	19.42	18.81
20	19.02	18.81	17.58	16.35
30	17.93	17.58	15.73	13.89
40	16.83	16.35	13.89	11.43
50	15.74	15.12	12.05	8.97
60	14.64	13.89	10.21	6.52
70	13.55	12.66	8.36	4.06
80	12.46	11.43	6.52	1.60

**4" x 0.030 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

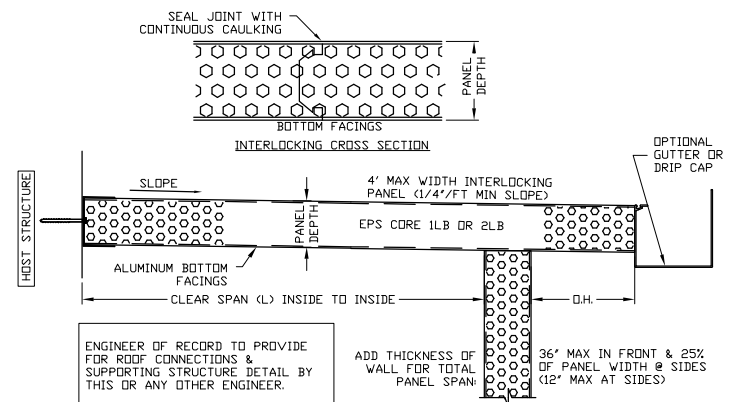
NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	24.17	24.17	24.17	24.17
20	23.64	23.64	23.41	23.11
30	22.57	22.57	21.90	21.01
40	21.51	21.51	20.39	18.91
50	20.45	20.45	18.88	16.80
60	19.39	19.39	17.37	14.70
70	18.33	18.33	15.86	12.59
80	17.26	17.26	14.35	10.49

**6" x 0.030 x 2 - LB EPS PANELS
(ALLOWABLE CLEAR SPAN CHARTS)**

NET ALLOWABLE LOAD (PSF) ¹	MAX. ALLOWABLE SPAN (FT)			
	L/80	L/120	L/180	L/240
10	24.00	24.00	24.00	23.84
20	23.65	23.65	23.34	22.84
30	22.94	22.94	22.59	21.85
40	22.23	22.23	21.85	20.85
50	21.53	21.53	21.10	19.86
60	20.82	20.82	20.36	18.87
70	20.11	20.11	19.61	17.87
80	19.40	19.40	18.87	16.88

GENERAL NOTES

- Composite panels shall be constructed using type 3003-H154 aluminum facings, 1 or 2 PCF ASTM C-578 Kingspan Insulation LLC or Imperial Foam & Insulation MFG. CO. brand EPS adhered to aluminum facings with Ashland Chemical 2020D ISO grip. Fabrication to be by Elite panel products only in accordance with approved fabrication methods.
- Elite roof panels maintain a UL 1715 (int) class 'B' (ext) rating and are NER-501 approved.
- This specification has been designed and shall be fabricated in accordance with the requirements of the Florida Building Code 8th Edition (FBC), composite panels comply with Chapter 7 Section 720, Chapter 8 Section 803, Class A interior finish, and Chapter 26 Section 2603. All local building code amendments shall be adhered to as required.
- The designer shall determine by accepted engineering practice the allowable loads for site specific load conditions (including load combinations) using the data from the allowable load tables and spans in this approval.
- Deflection limits and allowable spans have been listed to meet FBC including the HVHZ. In HVHZ, this product shall be used in structures "not to be considered living areas" per Section 1616 unless impact resistance in accordance to the HVHZ requirements are met.
- Safety factor of 2.0 has been used to develop allowable loads and spans from testing in accordance to the Guidelines for Aluminum Structures Part 1 and conforms to the FBC Chapter 16 and 20.
- Testing has been conducted in accordance to ASTM E72: Strength Test of Panels for Building Construction.
- Reference test reports: HETI-05-1988, HETI-06-2104, HETI-06-2066, HETI-06-2105, HETI-06-2067, HETI-05-1002, HETI-06-2107, HETI-05-1987, HETI-06-2069, HETI-06-2070, HETI-06-2071, HETI-05-1994, HETI-05-1991, HETI-06-2072, HETI-06-2073, HETI-06-2074, HETI-05-1996, HETI-05-1989, HETI-05-1993, HETI-05-1985, HETI-05-1995, HETI-05-1990, HETI-05-1997, HETI-05-2037, HETI-05-2029, HETI-05-2039, HETI-05-2030, HETI-05-2041, HETI-05-2048, HETI-05-2036, HETI-05-2031, HETI-05-2038, HETI-05-2065, HETI-05-2040, HETI-05-2042.
- Linear interpolation shall be allowed for figures within the tables shown.
- Panels with fan beams shall be considered equivalent to similar panels without fan beams. Design professionals may include the strength of the fan beam to exceed shown figures as part of site-specific engineering.



EPS ROOF PANEL/ SPAN DESCRIPTION

**DO KIM
& ASSOCIATES, LLC**
**CONSULTING
STRUCTURAL
ENGINEERS**

PO BOX 10039
Tampa, FL 33679
Tel: (813) 857-9555

Rev./Date	Description
8/15/2015	ISSUED FOR FBC 8th Edition PRODUCT APPROVAL
8/8/2020	ISSUED FOR FBC 7th Edition PRODUCT APPROVAL
6/15/2022	ADDED LABELING STATEMENT
8/15/2023	ISSUED FOR FBC 8th Edition PRODUCT APPROVAL

Elite Aluminum Corporation
4650 Lyons Technology Parkway
Coconut Creek, FL 33073

EPS FOAM CORE COMPOSITE PANELS
ALUMINUM/ALUMINUM SKIN
FLORIDA STATEWIDE PRODUCT APPROVAL

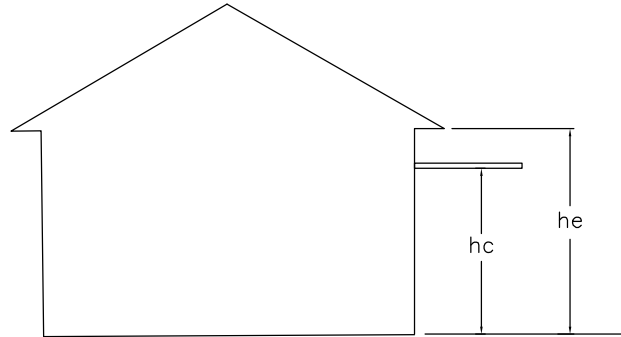
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CHECKED BY:	DYK
SCALE:	AS SHOWN
DATE:	2/19/12



Drawing No. - FL-1001
SHEET 1 OF 2

ELITE ALUMINUM PANELS ARE LABELED WITH A FL7561 LABEL TO ENSURE BUILDING INSPECTOR THAT THE INSULATED PANELS INSTALLED ARE APPROVED FOR USE IN THE STATE OF FLORIDA.

8th Edition FBC Basic Design Wind Speed and Allowable Design Wind Pressure for Attached Covers (canopies) on Buildings.



Attached Covers (canopies) on Buildings

1. Per 8th Edition FBC Chapter 16 for Components and Cladding Loads, ASCE/SEI 7-22 Chapter 30 for Components and Cladding for Attach Canopies on Buildings. Effective area for wind load calculations based on 10 sq. feet (absolute value of controlling design wind pressure is shown on span tables).
2. Use the wind load design pressures in the tables below for OPEN and ATTACHED covers (canopies) on buildings as a guide to determine allowable wind load design pressures. Use the design pressure selected to determine the allowable spans for the various panel types listed on Sheet 1.
2. The tables below ONLY applies to open and attached covers (canopies) on buildings per ASCE/SEI 7-22 Section 30.9 ATTACHED CANOPIES ON BUILDINGS and shall not be used for any other types of structures such as Enclosed, Freestanding Open, Partially Open, or Partially enclosed Buildings.
3. Roof covers attached to fascia are deemed $0.9 \leq hc/he \leq 1$.
4. Roof covers attached to the host structure underneath the fascia and overhang at deemed $0.5 \leq hc/he < 0.9$.

ASCE 7-22 Allowable Design Pressures			
ATTACHED TO FASCIA CANOPIES (Open Wind Flow), $0.9 \leq hc/he \leq 1$			
Wind Speed	Exposure B	Exposure C	Exposure D
110	10.7	15.98	19.36
120	12.8	19.02	23.04
130	15.0	22.32	27.04
140	17.4	25.88	31.37
150	19.9	29.71	36.01
160	22.7	33.81	40.97

ASCE 7-22 Allowable Design Pressures			
ATTACHED TO WALL CANOPIES (Open Wind Flow), $0.5 < hc/he < 0.9$			
Wind Speed	Exposure B	Exposure C	Exposure D
110	6.9	10.27	12.45
120	8.2	12.23	14.81
130	9.6	14.35	17.39
140	11.2	16.64	20.16
150	12.8	19.10	23.15
160	14.6	21.73	26.34

Notes:

1. The allowable design pressures listed in the tables are the absolute value of the controlling design pressure ($\pm$ dp).

DO KIM
& ASSOCIATES, LLC
CONSULTING
STRUCTURAL
ENGINEERS

PO BOX 10039
Tampa, FL 33679
Tel: (813) 857-9955

Rev./Date	Description
8/12/2015	ISSUED FOR FBC 8th Edition PRODUCT APPROVAL
8/8/2020	ISSUED FOR FBC 7th Edition PRODUCT APPROVAL
6/15/2022	ADDED LABELING STATEMENT
8/15/2023	ISSUED FOR FBC 8th Edition PRODUCT APPROVAL

Elite Aluminum Corporation
4650 Lyons Technology Parkway
Coconut Creek, FL 33073
EPS FOAM CORE COMPOSITE PANELS
ALUMINUM/ALUMINUM SKIN
FLORIDA STATEWIDE PRODUCT APPROVAL

DRAWN BY:	DYK
CHECKED BY:	DYK
SCALE:	AS SHOWN
DATE:	2/19/12



This document has been digitally signed and sealed by DO KIM & ASSOCIATES, LLC. The digital signature and seal are located at the bottom of the page. The digital signature and seal are not considered valid unless the signature and seal are located at the bottom of the page.

Drawing No. - FL-1001

SHEET 2 OF 2

Aluminum Beam

LIC# : KW-06019467, Build:20.24.12.02 (c) ENERCALC, LLC 1982-2025

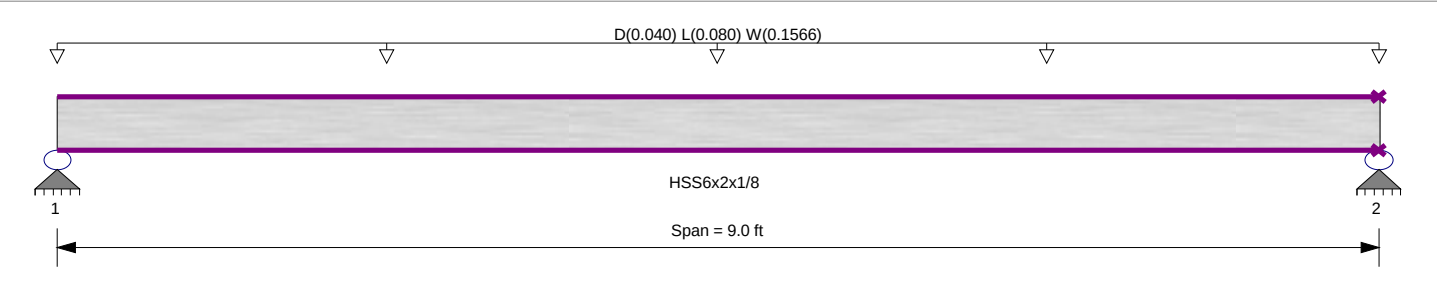
DESCRIPTION: Interior Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :Allowable Strength Design	Fy : Yield :	31.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	10,100.0 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 = 0.040, L = 0.080, W = 0.1566 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

				Design OK			
Maximum Bending Stress Ratio =		0.370 : 1		Maximum Shear Stress Ratio =		0.060 : 1	
Section used for this span		HSS6x2x1/8		Section used for this span		HSS6x2x1/8	
Ma : Applied		1.791 k-ft		Va : Applied		0.7962 k	
Mn / Omega : Allowable		4.840 k-ft		Vn/Omega : Allowable		13.308 k	
Load Combination		+D+0.750L+0.450W		Load Combination		+D+0.750L+0.450W	
Span # where maximum occurs		Span # 1		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		0.310 in Ratio = 348 >=240.		Span: 1 : W Only			
Max Upward Transient Deflection		0 in Ratio = 0 <240.0		n/a			
Max Downward Total Deflection		0.350 in Ratio = 308 >=180		Span: 1 : +D+0.750L+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	VnxVnx/Omega	
D Only													
Dsgn. L = 9.00 ft	1	0.097	0.016	0.47		0.47	8.08	4.84	1.00	1.00	0.21	22.22	13.31
+D+L													
Dsgn. L = 9.00 ft	1	0.265	0.043	1.28		1.28	8.08	4.84	1.00	1.00	0.57	22.22	13.31
+D+0.750L													
Dsgn. L = 9.00 ft	1	0.223	0.036	1.08		1.08	8.08	4.84	1.00	1.00	0.48	22.22	13.31
+D+0.60W													
Dsgn. L = 9.00 ft	1	0.294	0.047	1.42		1.42	8.08	4.84	1.00	1.00	0.63	22.22	13.31
+D+0.750L+0.450W													
Dsgn. L = 9.00 ft	1	0.370	0.060	1.79		1.79	8.08	4.84	1.00	1.00	0.80	22.22	13.31
+0.60D+0.60W													
Dsgn. L = 9.00 ft	1	0.255	0.041	1.23		1.23	8.08	4.84	1.00	1.00	0.55	22.22	13.31
+0.60D													
Dsgn. L = 9.00 ft	1	0.058	0.009	0.28		0.28	8.08	4.84	1.00	1.00	0.13	22.22	13.31

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+0.750L+0.450W	0.3501	4.526		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	0.0919	4.526		0.0000	7 of 43 0.000

Aluminum Beam

LIC# : KW-06019467, Build:20.24.12.02

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Interior Beam

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
+D+L	1	0.2502	4.526		0.0000	0.000
+D+0.750L	1	0.2107	4.526		0.0000	0.000
+D+0.60W	1	0.2779	4.526		0.0000	0.000
+D+0.750L+0.450W	1	0.3501	4.526		0.0000	0.000
+0.60D+0.60W	1	0.2411	4.526		0.0000	0.000
+0.60D	1	0.0552	4.526		0.0000	0.000
L Only	1	0.1583	4.526		0.0000	0.000
W Only	1	0.3099	4.526		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	0.796	0.796
Max Upward from Load Combinations	0.796	0.796
Max Upward from Load Cases	0.705	0.705
D Only	0.209	0.209
+D+L	0.569	0.569
+D+0.750L	0.479	0.479
+D+0.60W	0.632	0.632
+D+0.750L+0.450W	0.796	0.796
+0.60D+0.60W	0.548	0.548
+0.60D	0.125	0.125
L Only	0.360	0.360
W Only	0.705	0.705

Section Properties : HSS6x2x1/8

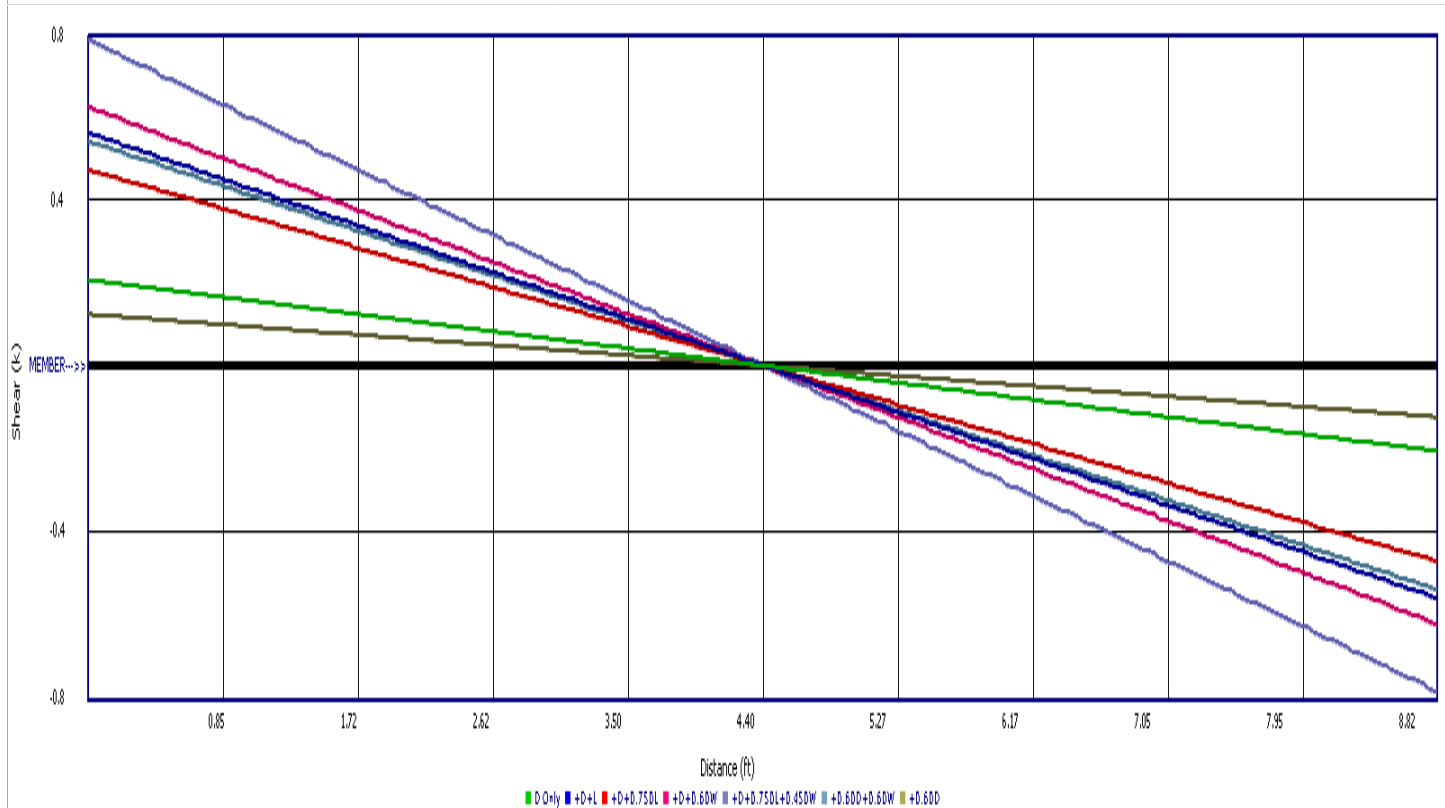
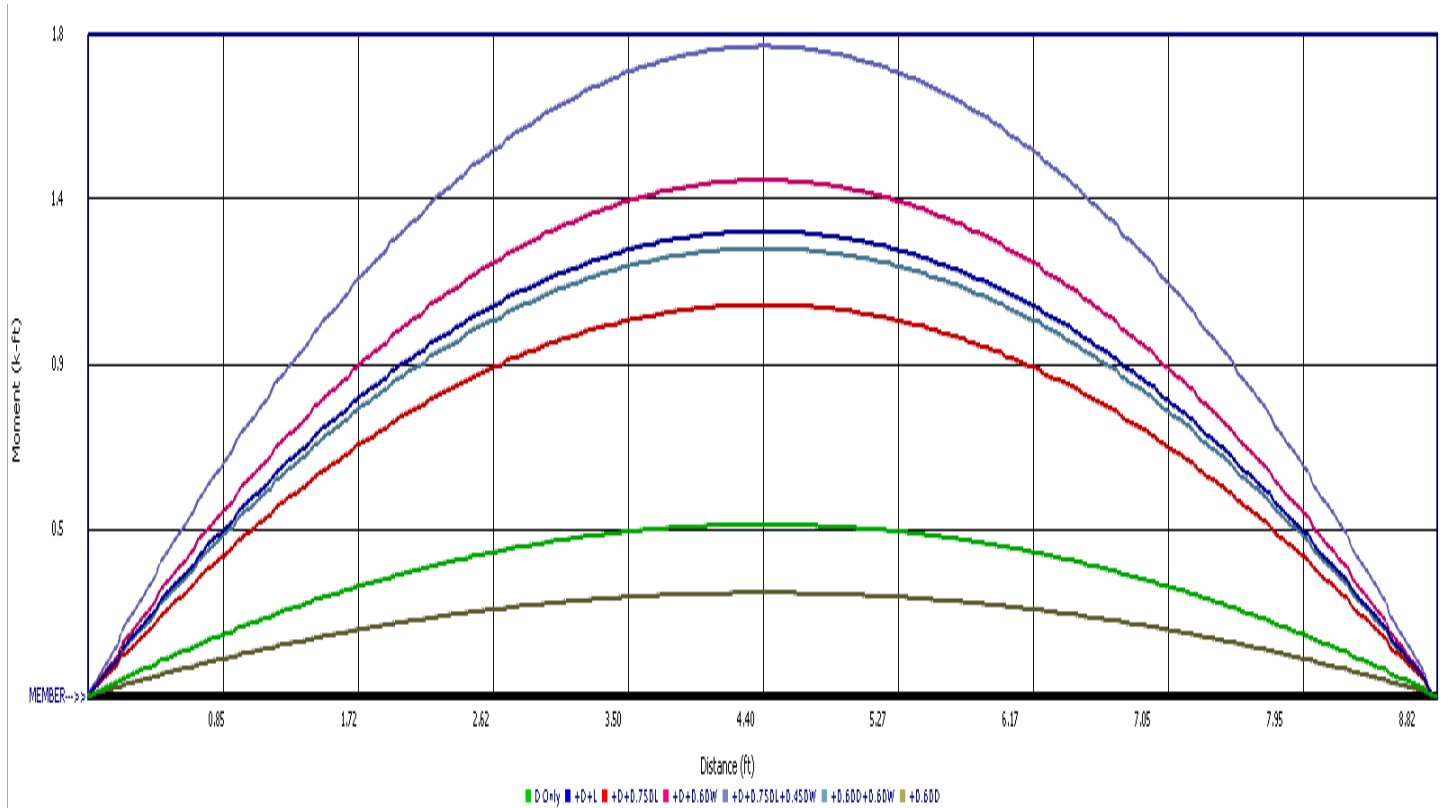
Depth	=	6.000 in	I xx	=	7.42 in^4	J	=	3.720 in^4
			S xx	=	2.47 in^3	Cw	=	2.57 in^6
Width	=	2.000 in	R xx	=	2.050 in			
Wall Thick	=	0.116 in	Zx	=	3.190 in^3			
Area	=	1.770 in^2	I yy	=	1.310 in^4	C	=	0.617 in^3
Weight	=	6.460 plf	S yy	=	1.310 in^3			
			R yy	=	0.861 in			
			Zy	=	1.460 in^3			
Ycg	=	3.000 in						

Aluminum Beam

LIC# : KW-06019467, Build:20.24.12.02

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Interior Beam

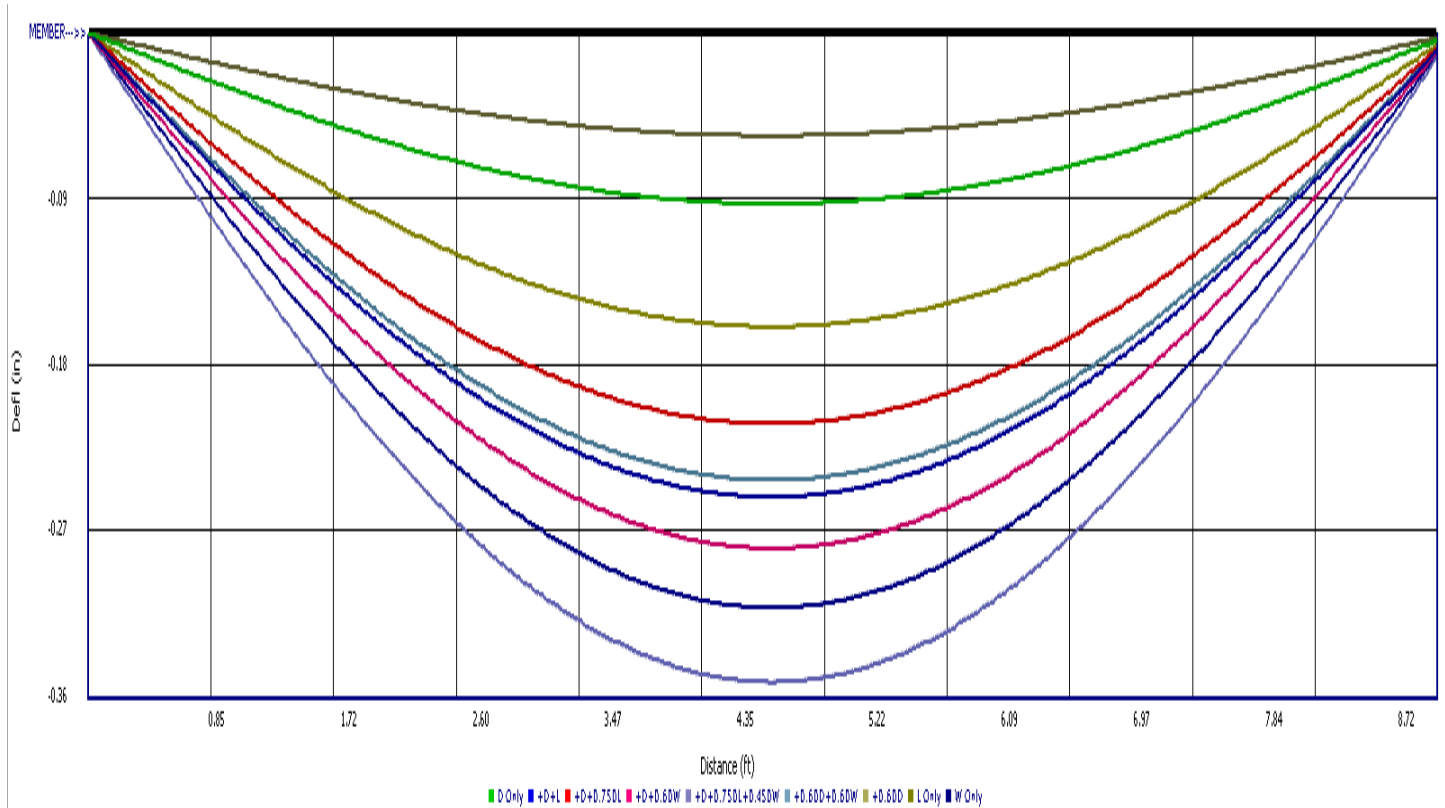


Aluminum Beam

LIC# : KW-06019467, Build:20.24.12.02

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Interior Beam



Project Title:
 Engineer:
 Project ID:
 Project Descr:

Aluminum Beam

Project File: 7465 SW 127 ST, PINECREST.ec6

LIC# : KW-06019467, Build:20.23.05.25

(c) ENERCALC INC 1983-2023

DESCRIPTION: Front Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

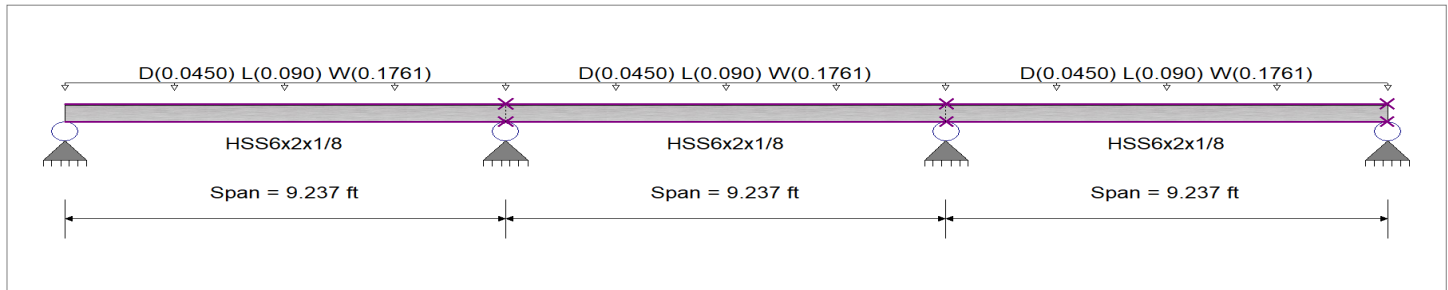
Analysis Method : Allowable Strength Design

Fy : Yield : 31.0 ksi

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

E: Modulus : 10,100.0 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 for Span Number 1

Uniform Load : D = 0.0450, L = 0.090, W = 0.1761 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.0450, L = 0.090, W = 0.1761 k/ft, Tributary Width = 1.0 ft

Load for Span Number 3

Uniform Load : D = 0.0450, L = 0.090, W = 0.1761 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.338 : 1	Maximum Shear Stress Ratio =		0.080 : 1
Section used for this span		HSS6x2x1/8	Section used for this span		HSS6x2x1/8
Ma : Applied		1.636 k-ft	Va : Applied		1.063 k
Mn / Omega : Allowable		4.840 k-ft	Vn/Omega : Allowable		13.308 k
Load Combination		+D+0.750L+0.450W	Load Combination		+D+0.750L+0.450W
Span # where maximum occurs		Span # 1	Location of maximum on span		9.237 ft
Span # where maximum occurs		Span # 1	Span # where maximum occurs		Span # 1
Maximum Deflection					
Max Downward Transient Deflection		0.206 in Ratio = 538	Span: 3 : W Only		
Max Upward Transient Deflection		-0.013 in Ratio = 8,433	Span: 3 : W Only		
Max Downward Total Deflection		0.224 in Ratio = 494	Span: 3 : +D+0.750L+0.450W		
Max Upward Total Deflection		-0.014 in Ratio = 7745	Span: 3 : +D+0.750L+0.450W		

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 = 9.24 ft	1	0.079	0.019	0.31	-0.38	0.38	8.08	4.84	1.00	1.00	0.25	22.22
Dsgn. L = 9.24 ft	2	0.079	0.019	0.10	-0.38	0.38	8.08	4.84	1.00	1.00	0.25	22.22
Dsgn. L = 9.24 ft	3	0.079	0.019	0.31	-0.38	0.38	8.08	4.84	1.00	1.00	0.25	22.22
+D+L												
Dsgn. L = 9.24 ft	1	0.238	0.056	0.92	-1.15	1.15	8.08	4.84	1.00	1.00	0.75	22.22
Dsgn. L = 9.24 ft	2	0.238	0.056	0.29	-1.15	1.15	8.08	4.84	1.00	1.00	0.75	22.22
Dsgn. L = 9.24 ft	3	0.238	0.056	0.92	-1.15	1.15	8.08	4.84	1.00	1.00	0.75	22.22
+D+0.750L												
Dsgn. L = 9.24 ft	1	0.198	0.047	0.77	-0.96	0.96	8.08	4.84	1.00	1.00	0.62	22.22
Dsgn. L = 9.24 ft	2	0.198	0.047	0.24	-0.96	0.96	8.08	4.84	1.00	1.00	0.62	22.22
Dsgn. L = 9.24 ft	3	0.198	0.047	0.77	-0.96	0.96	8.08	4.84	1.00	1.00	0.62	22.22
+D+0.60W												
Dsgn. L = 9.24 ft	1	0.266	0.063	1.03	-1.29	1.29	8.08	4.84	1.00	1.00	0.83	22.22

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Aluminum Beam

Project File: 7465 SW 127 ST, PINECREST.ec6

LIC# : KW-06019467, Build:20.23.05.25

(c) ENERCALC INC 1983-2023

DESCRIPTION: Front Beam

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	VnxVnx/Omega	
Dsgn. L = 9.24 ft	2	0.266	0.063	0.32	-1.29	1.29	8.08	4.84 1.00 1.00		0.83	22.22	13.31
Dsgn. L = 9.24 ft	3	0.266	0.063	1.03	-1.29	1.29	8.08	4.84 1.00 1.00		0.83	22.22	13.31
+D+0.750L+0.450W												
Dsgn. L = 9.24 ft	1	0.338	0.080	1.31	-1.64	1.64	8.08	4.84 1.00 1.00		1.06	22.22	13.31
Dsgn. L = 9.24 ft	2	0.338	0.080	0.41	-1.64	1.64	8.08	4.84 1.00 1.00		1.06	22.22	13.31
Dsgn. L = 9.24 ft	3	0.338	0.080	1.31	-1.64	1.64	8.08	4.84 1.00 1.00		1.06	22.22	13.31
+0.60D+0.60W												
Dsgn. L = 9.24 ft	1	0.234	0.055	0.91	-1.13	1.13	8.08	4.84 1.00 1.00		0.74	22.22	13.31
Dsgn. L = 9.24 ft	2	0.234	0.055	0.28	-1.13	1.13	8.08	4.84 1.00 1.00		0.74	22.22	13.31
Dsgn. L = 9.24 ft	3	0.234	0.055	0.91	-1.13	1.13	8.08	4.84 1.00 1.00		0.74	22.22	13.31
+0.60D												
Dsgn. L = 9.24 ft	1	0.048	0.011	0.18	-0.23	0.23	8.08	4.84 1.00 1.00		0.15	22.22	13.31
Dsgn. L = 9.24 ft	2	0.048	0.011	0.06	-0.23	0.23	8.08	4.84 1.00 1.00		0.15	22.22	13.31
Dsgn. L = 9.24 ft	3	0.048	0.011	0.18	-0.23	0.23	8.08	4.84 1.00 1.00		0.15	22.22	13.31

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.450W	1	0.2242	4.126		0.0000	0.000
+D+0.750L+0.450W	2	0.0159	4.680	+D+0.750L+0.450W	-0.0143	1.047
+D+0.750L+0.450W	3	0.2224	5.172		0.0000	1.047

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.708	1.948	1.948	0.708
Max Upward from Load Combinations	0.708	1.948	1.948	0.708
Max Upward from Load Cases	0.651	1.789	1.789	0.651
D Only	0.166	0.457	0.457	0.166
+D+L	0.499	1.372	1.372	0.499
+D+0.750L	0.416	1.143	1.143	0.416
+D+0.60W	0.557	1.531	1.531	0.557
+D+0.750L+0.450W	0.708	1.948	1.948	0.708
+0.60D+0.60W	0.490	1.348	1.348	0.490
+0.60D	0.100	0.274	0.274	0.100
L Only	0.333	0.914	0.914	0.333
W Only	0.651	1.789	1.789	0.651

Section Properties : HSS6x2x1/8

Depth	=	6.000 in	I xx	=	7.42 in^4	J	=	3.720 in^4
			S xx	=	2.47 in^3	Cw	=	2.57 in^6
Width	=	2.000 in	R xx	=	2.050 in			
Wall Thick	=	0.116 in	Zx	=	3.190 in^3			
Area	=	1.770 in^2	I yy	=	1.310 in^4	C	=	0.000 in^3
Weight	=	6.448 plf	S yy	=	1.310 in^3			
			R yy	=	0.861 in			
			Zy	=	1.460 in^3			
Ycg	=	3.000 in						

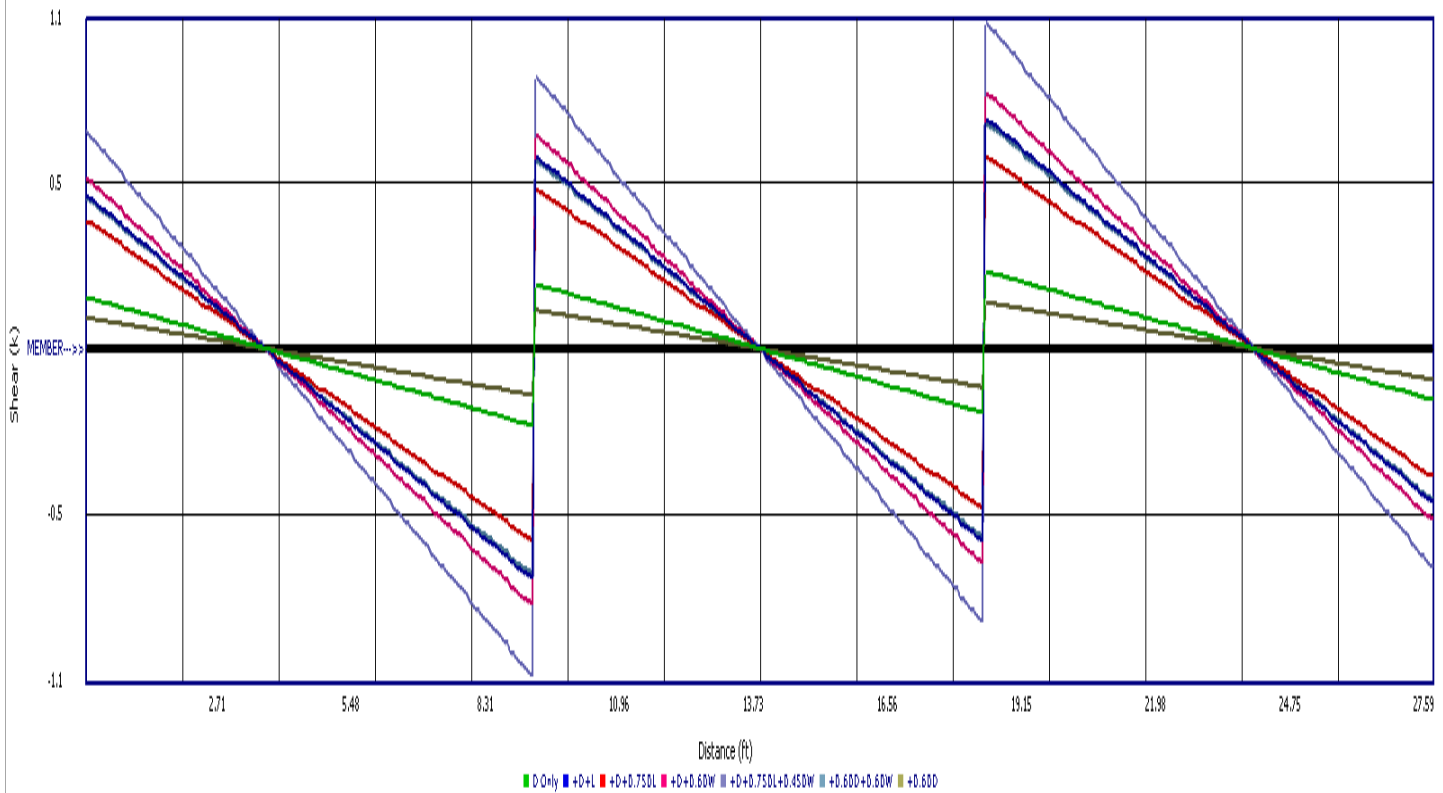
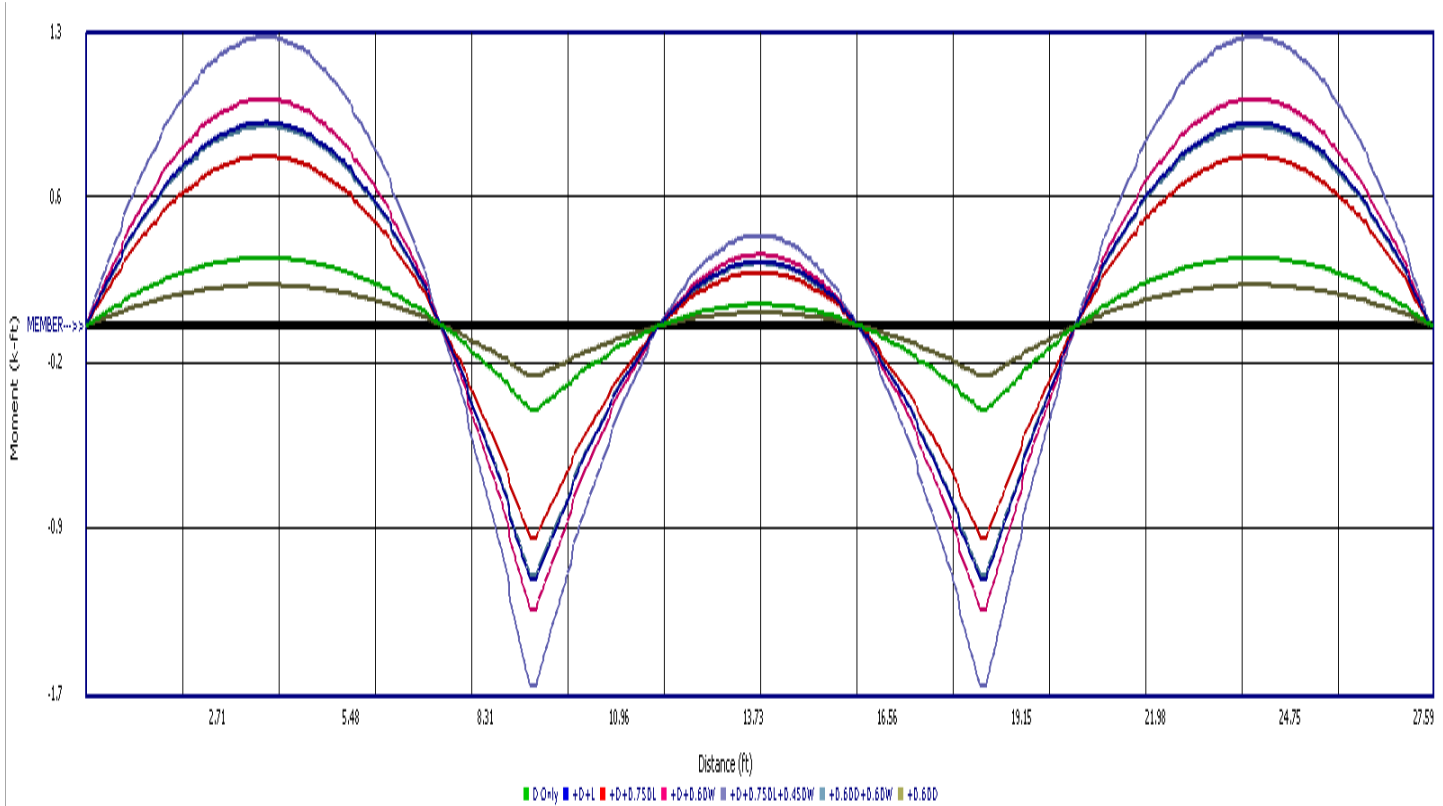
Aluminum Beam

Project File: 7465 SW 127 ST, PINECREST.ec6

LIC# : KW-06019467, Build:20.23.05.25

(c) ENERCALC INC 1983-2023

DESCRIPTION: Front Beam



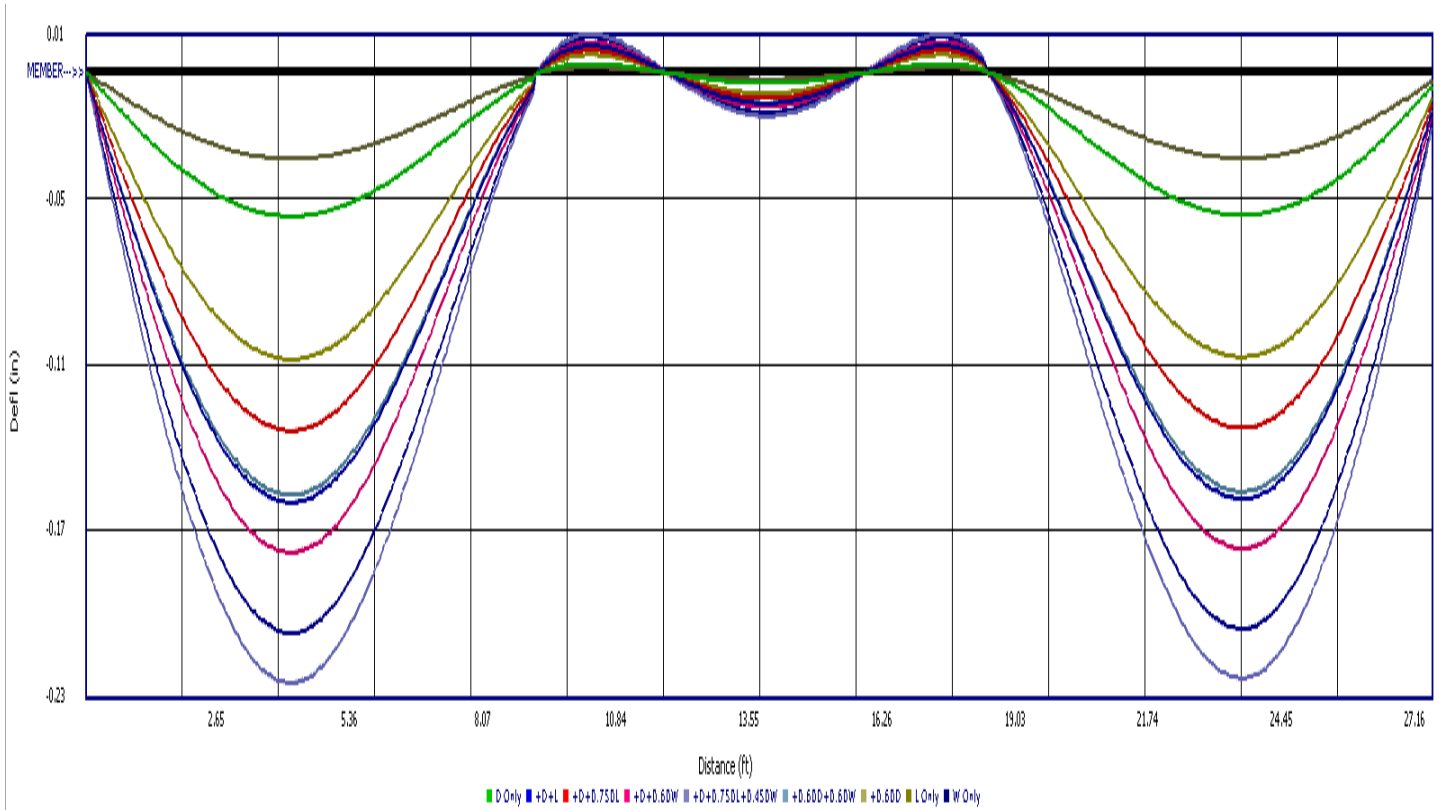
Aluminum Beam

Project File: 7465 SW 127 ST, PINECREST.ec6

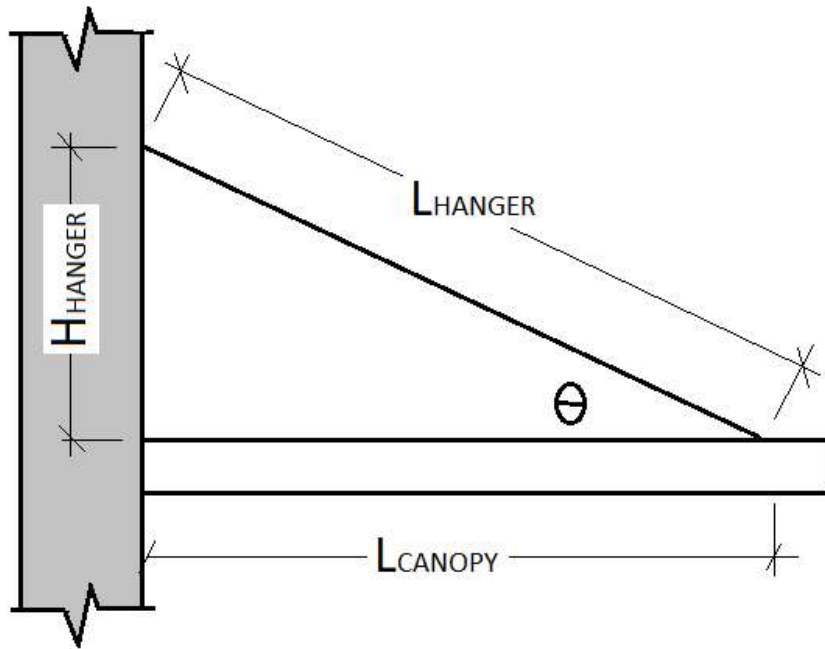
LIC# : KW-06019467, Build:20.23.05.25

(c) ENERCALC INC 1983-2023

DESCRIPTION: Front Beam



Aluminum Hanger 2x2x1/8 Design



$$\theta := 24.0 \quad \text{Degrees}$$

$$L_{\text{Canopy}} := 9.0 \quad (\text{ft})$$

$$H_{\text{Hanger}} := 4.0 \quad (\text{ft})$$

$$L_{\text{Hanger}} := 9.3 \quad (\text{ft})$$

Loads Data:

$$P := 1.948 \quad \text{Axial in Column (kips)}$$

$$M_x := 0.00 \quad \text{X-X Axis Moments (kips-ft)}$$

$$M_y := 0.00 \quad \text{Y-Y Axis Moments (kips-ft)}$$

**For Rect. Tubes & Struct. Shapes
For Temper Designation T-5, T-6**

General Data:

$$L_b := L_{\text{Hanger}} \quad \text{Unbraced Length of Column (ft)}$$

$$K := 1.00 \quad \text{Effective Length Factor}$$

$$K_c := 1.12 \quad \text{Coefficient for Compression Members}$$

$$\quad K_c = 1.12 \quad \text{Non welded}$$

$$\quad K_c = 1.10 \quad \text{Welded}$$

$$K_t := 1.00 \quad \text{Coefficient for tension Members } K_t = 1.00$$

$$n_u := 1.95$$

$$n_y := 1.65 \quad \text{Safety Factors}$$

$$n_a := 1.20$$

$$C_{mx} := 0.85 \quad \text{Reduction Coefficient}$$

$$\quad C_m = 0.85 \quad \text{if Not Sidesway \& Ends Restrained against Rotation}$$

$$C_{my} := 0.85 \quad \text{Cm=1 if Not Sideway \& Ends Unrestrained against Rotation}$$

Column Material Data:

$$E := 10000.00 \quad \text{Aluminum Modulus of Elasticity (Ksi)}$$

$$F_{cy} := 40.00 \quad \text{Compressive Yield Strength of Aluminum Column (Ksi)}$$

$$F_{ty} := 40.00 \quad \text{Tensile Yield Strength (ksi)}$$

$$F_{tu} := 45.00 \quad \text{Tensile Ultimate Strength (ksi)}$$

Section Data:

$$I_x := 0.552 \quad \text{Moment of Inertia of Column X-X Axis Around Strong Axis (in}^4\text{)}$$

$$I_y := 0.552 \quad \text{Moment of Inertia of Column Y-Y Axis Around Weak Axis (in}^4\text{)}$$

$$A := 0.938 \quad \text{Area of Column (in}^2\text{)}$$

$$S_x := 0.552 \quad \text{Elastic Modulus Section X-X Axis Around Strong Axis (in}^3\text{)}$$

$$S_y := 0.552 \quad \text{Elastic Modulus Section Y-Y Axis Around Weak Axis (in}^3\text{)}$$

Formulas for Buckling Constants for Columns & Beams Flanges

$$B_c := F_{cy} \left[1 + \left(\frac{F_{cy}}{2250} \right)^{\frac{1}{2}} \right] \quad B_c = 45.333 \quad \text{ksi}$$

$$D_c := \frac{B_c}{10} \left(\frac{B_c}{E} \right)^{\frac{1}{2}} \quad D_c = 0.305 \quad \text{ksi}$$

$$C_c := 0.41 \cdot \frac{B_c}{D_c} \quad C_c = 60.894$$

$$S_1 := \frac{B_c - \frac{n_u \cdot F_{cy}}{K_c \cdot n_y}}{D_c} \quad S_1 = 10.24$$

$$S_2 := C_c \quad S_2 = 60.894$$

Bending Around Strong Axis X-X

$$F_{bx} := \min \left(\frac{F_{ty}}{n_y}, \frac{F_{tu}}{K_t \cdot n_u} \right) \quad F_{bx} = 23.077 \quad \text{ksi}$$

Bending Around Weak Axis Y-Y

$$F_{by} := \min \left(\frac{1.3F_{ty}}{n_y}, \frac{1.42F_{tu}}{K_t \cdot n_u} \right) \quad F_{by} = 31.515 \quad \text{ksi}$$

Check Axial Stress

$$r_x := \sqrt{\frac{I_x}{A}} \quad r_x = 0.767 \quad \text{Radius of Gyration X-X Axis (in)}$$

$$r_y := \sqrt{\frac{I_y}{A}} \quad r_y = 0.767 \quad \text{Radius of Gyration Y-Y Axis (in)}$$

$$r := \min(r_x, r_y)$$

$$r = 0.767 \quad \text{Radius of Gyration Minimum (in)}$$

$$SR := \frac{K \cdot (L_b \cdot 12)}{r}$$

$$SR = 145.478$$

$$\text{SLENDERNESS} := \text{if}(SR \leq 200, \text{"OK"}, \text{"NG"})$$

$$\boxed{\text{SLENDERNESS} = \text{"OK"}}$$

$$F_a := \begin{cases} \frac{F_{cy}}{K_c \cdot n_y} & \text{if } SR \leq S_1 \\ \frac{1}{n_u} \cdot (B_c - D_c \cdot SR) & \text{if } S_1 < SR < S_2 \\ \frac{\pi^2 \cdot E}{n_u \cdot SR^2} & \text{if } SR \geq S_2 \end{cases} \quad F_a = 2.392 \quad \text{Allowable Compression Stress (ksi)}$$

$$f_a := \frac{P}{A} \quad f_a = 2.077 \quad \text{ksi}$$

$$AXIAL := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } f_a \leq F_a \end{cases} \quad \boxed{AXIAL = \text{"OK"}}$$

Check Moment Stress

$$f_{bx} := \frac{M_x \cdot 12}{S_x} \quad f_{bx} = 0 \quad \text{ksi} \quad f_{by} := \frac{M_y \cdot 12}{S_y} \quad f_{by} = 0 \quad \text{ksi}$$

Check Strength Interaction Equation

$$F'_{ex} := \frac{\pi^2 \cdot E}{n_u \left(\frac{K \cdot L_b \cdot 12}{r_x} \right)^2} \quad F'_{ex} = 2.392 \quad \text{ksi}$$

$$F'_{ey} := \frac{\pi^2 \cdot E}{n_u \left(\frac{K \cdot L_b \cdot 12}{r_y} \right)^2} \quad F'_{ey} = 2.392 \quad \text{ksi}$$

$$IE_{\text{strength}} := \begin{cases} \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} & \text{if } \frac{f_a}{F_a} \leq 0.15 \\ \frac{f_a}{F_a} + \frac{C_{mx} \cdot f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}} \right) \cdot F_{bx}} + \frac{C_{my} \cdot f_{by}}{\left(1 - \frac{f_a}{F'_{ey}} \right) \cdot F_{by}} & \text{if } \frac{f_a}{F_a} > 0.15 \end{cases} \quad IE_{\text{strength}} = 0.868$$

$$BENDING_{\text{strength}} := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } IE_{\text{strength}} \leq 1 \end{cases} \quad \boxed{BENDING_{\text{strength}} = \text{"OK"}}$$



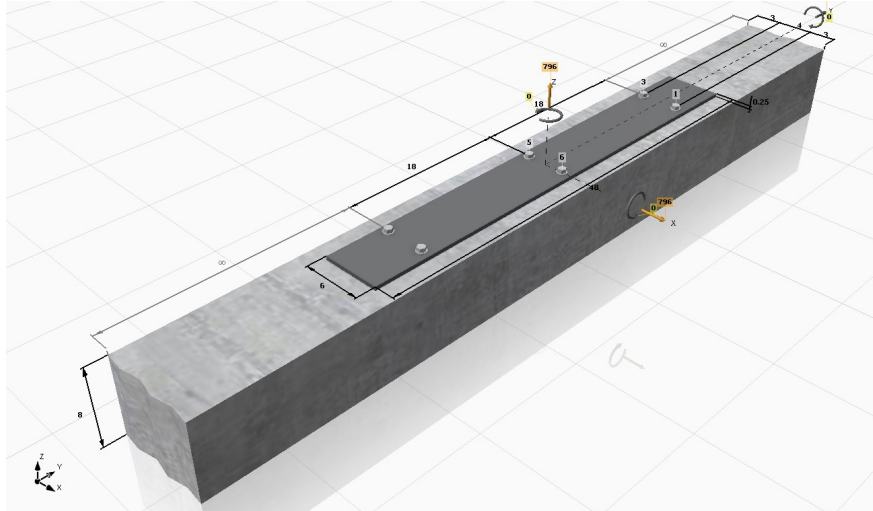
03 Bolts

6/17/2024

1. Project Information**Company:****Project Engineer:** -**Address:** -**Phone:** -**Email:** -**Project Name:** -**Project Address:** -**Notes:**

2. Selected Anchor Information**Selected Anchor :** Screw-Bolt+**Brand:** DEWALT**Material:** 1/2" Ø Low Carbon Steel**Embedment:** h_{ef} 1.75 in h_{nom} 2.5 in**Approval:** ICC-ES ESR-3889**Issued/Revision:** Nov, 2024 -**Drill method:** Hammer Drilled**3. Design Principles****Design Method:** ACI 318-14**Load Combinations:** Section 5.3 User Defined Loads**4. Base Material Information****Concrete:****Type:** Uncracked Normal Weight Concrete**Strength** 3000 psi**Reinforcement:****Supplemental Reinforcement** False**Spacing** Tension No (Condition B) Shear No (Condition B)**Controls Breakout** Tension False Shear False**Base Plate:****Sizing** Thickness 0.25 in Length 48 in Width 6 in**Standoff** None Height 0 in**Strength** 36000 psi**Profile:** None

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



h_{slab}	8	in	h_{min}	4.000	in
Edge Cx-	3	in	c_{min}	1.750	in
Edge Cx+	3	in	c_{ac}	3.300	in
Edge Cy-	∞	in	s_{min}	2.750	in
Edge Cy+	∞	in			

6. Summary Results

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	132.67	13309.00	0.010	OK	
Concrete Breakout Strength:	796.00	11881.00	0.067	OK	Controls

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	133.00	5316.00	0.025	OK	
Concrete Breakout Strength:	398.00	5321.00	0.075	OK	Controls
Pryout Strength	796.00	12795.00	0.062	OK	

19 of 48

7.Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

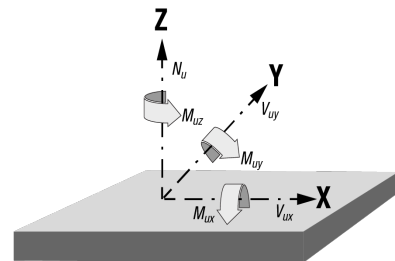


- The results of the calculations carried out by means of the DDA Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an design professional/engineer, particularly with regard to compliance with applicable standards, norms and permits, prior to using them for your specific project. The DDA Software serves only as an aid to interpret standards, norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.

8.Load Condition

Design Loads / Actions

Nu	796.0	lb	Vux	796.0	lb	Vuy	0.0	lb
Muz	0.0	in-lb	Mux	0.0	in-lb	Muy	0.0	in-lb
Consider Load Reversal			X Direction	0%		Y Direction	0%	



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



03 Bolts

6/17/2024

9.Load Distribution

Max. concrete compressive strain:	0.000	%	<u>Anchor Eccentricity</u>					
Max. concrete compressive stress:	0.000	psi	ex	0	in	ey	0	in
Resulting tension force:	796.000	lb	<u>Profile Eccentricity</u>					
Resulting compression force:	0.000	lb	ex	0	in	ey	0	in

Resulting anchor forces / Load distribution

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb)		Anchor Coordinates (in)	
			Shear X	Shear Y	X	Y
1	132.67	132.7	132.7	0.0	2.000	18.000
2	132.67	132.7	132.7	0.0	2.000	-18.000
3	132.67	132.7	132.7	0.0	-2.000	18.000
4	132.67	132.7	132.7	0.0	-2.000	-18.000
5	132.67	132.7	132.7	0.0	-2.000	0.000
6	132.67	132.7	132.7	0.0	2.000	0.000

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



10.Design Proof Tension Loading

Steel Strength

ACI 318-14 17.4.1

Variables

N_{sa} (lb)	ϕ
20475.000	0.65

Results

ϕN_{sa}	=	13309.0	lb
N_{ua}	=	132.7	lb
Utilization	=	1.0%	

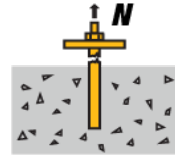


Table 17.3.1.1

Concrete Breakout Strength

ACI 318-14 17.4.2

Equations

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

Eqn. 17.4.2.1b

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

Eqn. 17.4.2.2a

Variables

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
145.688	27.562	1.000	1.000	1.000	0.909
c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
3.300	30.000	1.000	3000	1.75	3
N_b (lb)	ϕ				
3803.990	0.65				

Results

ϕN_{cbg}	=	11881	lb
N_{ua}	=	796.0	lb
Utilization	=	6.7%	

Table 17.3.1.1





11.Design Proof Shear Loading

Reference

Steel Strength

ACI 318-14 17.5.1

Variables

V_{sa} (lb)	ϕ
8860.00	0.60

Results

ϕV_{sa}	=	5316	lb
V_{ua}	=	133	lb
Utilization	=	2.5%	

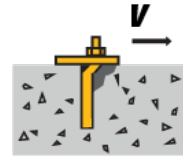


Table 17.3.1.1

Concrete Breakout Strength

ACI 318-14 17.5.2

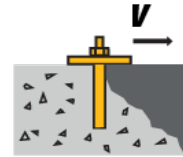
Equations

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

Eqn. 17.5.2.1b

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

Eqn. 17.5.2.2a



Variables

A_{Vc} (in ²)	A_{Vc0} (in ²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
121.500	40.500	1.000	1.000	1.400	1.000
$\Psi_{a,V}$	l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{al} (in)
1.000	1.750	0.500	1.000	3000.000	3.000
V_b (lb)	ϕ				
1809.836	0.70				
		V_{cbg} (lb)			
		7601.311			

Results

ϕV_{cbg}	=	5321	lb
Direction	=	X+	
V_{ua}	=	398	lb
Utilization	=	7.5%	

Table 17.3.1.1

Shear Breakout Case 1 Controls

**Pryout Strength****ACI 318-14 17.5.3****Equations**

$$V_{cpg} = k_{cp} \cdot N_{cpg}$$

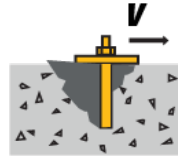
Eqn. 17.5.3.1b

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

Eqn. 17.4.2.1b

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

Eqn. 17.4.2.2a

**Variables**

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
145.688	27.563	1.000	1.000	1.000	0.909
c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
3.300	30.000	1.000	1.75	3000.000	3.000
N_b (lb)	k_{cp}	N_{cpg} (lb)	ϕ		
3803.986	1.000	18278.896	0.70		

Results

ϕV_{cpg}	=	12795.00 lb
V_{ua}	=	796 lb
Utilization	=	6.2%

Table 17.3.1.1

12. Interaction of Tension and Shear Loads

Reference

ACI 318-14 17.6

Equations

$$(N_{ua} / \phi \cdot N_n)^{5/3} + (V_{ua} / \phi \cdot V_n)^{5/3} \leq 1.0$$

Eqn. 17.6.3

Variables

$N_{ua} / \phi \cdot N_n$	$V_{ua} / \phi \cdot V_n$
0.067	0.075

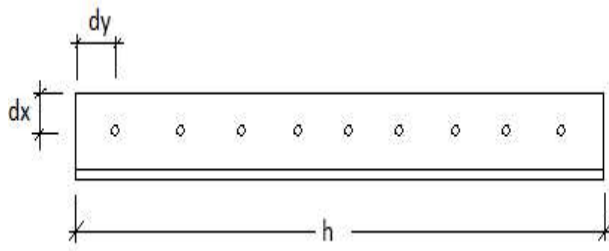
Results

0.024	$\leq$	1.0
Status	:	OK

ANCHOR DESIGN CRITERIA IS SATISFIED



3"x3"x1/4" Angle 3" L.G.



General Data:

$D_L := 10.0$ Gravity Load on Connection (lbs)
 $V_{conn} := 1948.0$ Shear on Connection (lbs)
 $T_t := 1948.0$ Tension Load on Connection (lbs)
 $GAP := 3.50$ Maximum dist. from base Plate to load (in)

Anchors Locations Data:

$d_x := 1.5$ Distance from Edge of Plate to Center of Anchor Along XX (in)
 $d_y := 1.5$ Distance from Edge of Plate to Center of Anchor Along YY (in)

Angle Material Data:

$b := 3.0$	Angle Width (in)	$d_b := 0.625$	Bolt diam (in)
$L := 3.0$	Angle Length (in)	$U := 0.750$	Reduction Factor
$thk := 0.25$	Angle Thickness (in)	$l_c := 3.00$	Spacing (in)
$h := 3.0$	Angle Length (in)	$\Omega := 2.00$	(Safety factor)
$e_v := GAP - thk$	$e_v = 3.25$		in

Angle Material Data:

$F_b := 19000.0$ Angle Allowabel Bending Stress (psi)
 $F_v := 5000.0$ Angle Allowabel Shear Stress (psi)
 $F_{ty} := 15000.0$ Angle Allowable Tension Stress for yielding (psi)
 $F_{tf} := 15000.0$ Angle Allowable Tension Stress for fracture (psi)
 $F_y := 24000.0$ Angle Yielding Stress (psi)
 $F_u := 30000.0$ Angle Ultimate Stress (psi)

Shear & Moment per angle:

$$V_{\text{total}} := \frac{D_L}{2} + \frac{V_{\text{conn}}}{2}$$

$$V_{\text{total}} = 979.00 \quad \text{lbs}$$

$$M_V := \frac{(V_{\text{total}} \cdot e_v)}{2}$$

$$M_V = 1590.88 \quad \text{lbs} - \text{in}$$

$$S_V := \frac{\text{thk} \cdot (h)^2}{6}$$

$$S_V = 0.38 \quad \text{in}^3$$

$$A_V := b \cdot h$$

$$A_V = 9.00 \quad \text{in}^2$$

$$A_n := \text{thk} \cdot h - \left[3 \cdot \left[\frac{(3.1415 \cdot d_b^2)}{4} \right] \right]$$

$$A_n = -0.17 \quad \text{in}^2$$

Check Bending Stress :

$$fb_V := \frac{M_V}{S_V}$$

$$fb_V = 4242.33 \quad \text{psi}$$

$$\text{BENDING}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } F_b \geq fb_V \end{cases}$$

$$\boxed{\text{BENDING}_V = \text{"OK"}}$$

Check Tension Stress :

$$f_{ty} := \frac{(T_t)}{0.6 \cdot A_V} \quad \text{for yielding} \quad f_{ty} = 360.74 \text{ psi}$$

$$f_{tf} := \frac{(T_t)}{0.5 \cdot (U \cdot A_n)} \quad \text{for fracture} \quad f_{tf} = -3049 \text{ psi}$$

$$\text{TENSION}_{\text{yielding}} := \text{if}(F_{ty} \geq f_{ty}, \text{"OK"}, \text{"NG"})$$

$$\boxed{\text{TENSION}_{\text{yielding}} = \text{"OK"}}$$

$$\text{TENSION}_{\text{fracture}} := \text{if}(F_{tf} \geq f_{tf}, \text{"OK"}, \text{"NG"})$$

$$\boxed{\text{TENSION}_{\text{fracture}} = \text{"OK"}}$$

Check Shear Stress :

$$fv_V := \frac{V_{\text{total}}}{A_V}$$

$$fv_V = 108.78 \quad \text{psi}$$

$$\text{SHEAR}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } F_v \geq fv_V \end{cases}$$

$$\boxed{\text{SHEAR}_V = \text{"OK"}}$$

Check Combined Bending & Shear :

$$IE_V := \frac{fb_V}{F_b} + \frac{fv_V}{F_v} + \max\left(\frac{f_{ty}}{F_{ty}}, \frac{f_{tf}}{F_{tf}}\right)$$

$$IE_V = 0.27$$

$$\text{COMBINED}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } 1 \geq IE_V \end{cases}$$

$$\boxed{\text{COMBINED}_V = \text{"OK"}}$$

Check Horizontal Plate Stress

Check at Plane of Failure for Moment :

$$b_{\text{plane}} := \text{if} \left(L \geq b, \sqrt{L^2 + b^2}, \sqrt{2 \cdot L^2} \right) \quad b_{\text{plane}} = 4.24 \quad \text{in}$$

$$d_{\text{plane}} := \sqrt{(b)^2 - \left(\frac{L}{2} \right)^2} \quad d_{\text{plane}} = 2.60 \quad \text{in}$$

$$d_{\text{screws}} := d_{\text{plane}} - \sqrt{d_x^2 + d_y^2} \quad d_{\text{screws}} = 0.48 \quad \text{in}$$

$$S_h := \frac{b_{\text{plane}} \cdot h^2}{6} \quad S_v = 0.38 \quad \text{in}^3$$

Check Bending Stress :

$$fb_h := \frac{M_v}{S_h} \quad fb_h = 249.98 \quad \text{psi}$$

$$\text{BENDING}_h := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_b \geq fb_h \end{cases} \quad \boxed{\text{BENDING}_h = \text{"OK"}}$$

Check at Plane of Failure for Moment :

$$V_h := V_{\text{total}} \quad V_h = 979.00 \quad \text{lbs}$$

$$A_h := b \cdot h \quad A_h = 9.00 \quad \text{in}^2$$

Check Shear Stress :

$$fv_h := \frac{V_h}{A_h} \quad fb_h = 249.98 \quad \text{psi}$$

$$\text{SHEAR}_h := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq fv_h \end{cases} \quad \boxed{\text{SHEAR}_h = \text{"OK"}}$$



10/18/2024

1. Project Information

Company:
Project Engineer: -

Address: -

Phone: -

Email: -

Project Name: -

Project Address: -

Notes:

2. Selected Anchor Information

Selected Anchor : Screw-Bolt+

Brand: DEWALT

Material: 5/8" Ø Low Carbon Steel

Embedment: h_{ef} 2.24 in h_{nom} 3.25 in

Approval: ICC-ES ESR-3889

Issued/Revision: Nov,2024 -

Drill method: Hammer Drilled


3. Design Principles

Design Method: ACI 318-14

Load Combinations: Section 5.3 User Defined Loads

4. Base Material Information

Concrete:

Type: Cracked Normal Weight Concrete

Strength 3000 psi

Reinforcement:

Edge Reinforcement None or < #4 Rebar

Spacing Tension No (Condition B) Shear No (Condition B)

Controls Breakout Tension False Shear False

Base Plate:

Sizing Thickness 0.25 in Length 8 in Width 3 in

Standoff None Height 0 in

Strength 36000 psi

Profile: None

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

A 3D model of a mechanical part, likely a bracket or support, with the following dimensions and features:

- Overall Dimensions:**
 - Length: 100
 - Width: 8
 - Height: 8
- Top Surface Features:**
 - A central rectangular plate with a width of 6 and a length of 8.
 - Two circular holes on the plate, each with a diameter of 0.25.
 - A central vertical slot with a width of 4.
- Coordinate Systems:**
 - Top Center:** A coordinate system with the Z-axis pointing upwards (labeled 1948) and the Y-axis pointing to the right (labeled 0).
 - Right Side:** A coordinate system with the X-axis pointing to the right (labeled 1948) and the Y-axis pointing upwards (labeled 0).
- Curved End:** The right end of the part is curved, with a radius of 4.

h_{slab}	8	in	h_{min}	5.000	in
Edge Cx-	4	in	c_{min}	1.750	in
Edge Cx+	4	in	c_{ac}	6.300	in
Edge Cy-	∞	in	s_{min}	2.750	in
Edge Cy+	∞	in			

Tension Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	974.00	17069.00	0.057	OK	Controls
Concrete Breakout Strength:	1948.00	4371.00	0.446	OK	
Pullout Strength:	974.00	2193.00	0.444	OK	

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	974.00	7386.00	0.132	OK	
Concrete Breakout Strength:	1948.00	3104.00	0.628	OK	Controls
Pryout Strength	1948.00	4708.00	0.414	OK	

30 of 43

7. Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

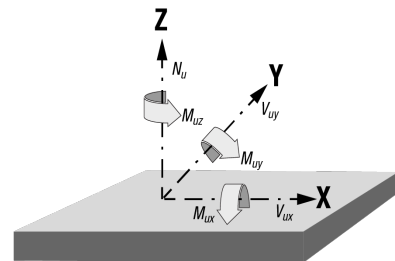


- The results of the calculations carried out by means of the DDA Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an design professional/engineer, particularly with regard to compliance with applicable standards, norms and permits, prior to using them for your specific project. The DDA Software serves only as an aid to interpret standards, norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.

8. Load Condition

Design Loads / Actions

Nu	1948.0	lb	Vux	1948.0	lb	Vuy	0.0	lb
Muz	0.0	in-lb	Mux	0.0	in-lb	Muy	0.0	in-lb
Consider Load Reversal			X Direction	0%		Y Direction	0%	





10/18/2024

9. Load Distribution

Max. concrete compressive strain:	0.000	%	<u>Anchor Eccentricity</u>					
Max. concrete compressive stress:	0.000	psi	ex	0	in	ey	0	in
Resulting tension force:	1948.000	lb	<u>Profile Eccentricity</u>					
Resulting compression force:	0.000	lb	ex	0	in	ey	0	in
<u>Resulting anchor forces / Load distribution</u>								

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb)		Anchor Coordinates (in)	
			Shear X	Shear Y	X	Y
1	974.00	974.0	974.0	0.0	0.000	2.500
2	974.00	974.0	974.0	0.0	0.000	-2.500

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



10/18/2024

10.Design Proof Tension Loading

Steel Strength

ACI 318-14 17.4.1

Variables

N_{sa} (lb)	ϕ
26260.000	0.65

Results

ϕN_{sa}	=	17069.0	lb
N_{ua}	=	974.0	lb
Utilization	=	5.7%	



Table 17.3.1.1

Concrete Breakout Strength

ACI 318-14 17.4.2

Equations

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

Eqn. 17.4.2.1b

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

Eqn. 17.4.2.2a

Variables

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
78.758	45.158	1.000	1.000	1.000	1.000
c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
6.300	21.000	1.000	3000	2.24	4
N_b (lb)	ϕ				
3856.130	0.65				

Results

ϕN_{cbg}	=	4371	lb
N_{ua}	=	1948.0	lb
Utilization	=	44.6%	

Table 17.3.1.1



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

Pullout Strength:

ACI 318-14 17.4.3

Equations

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



Eqn. 17.4.3.1

Variables

$\Psi_{c,P}$	$N_{p,cr}$ (lb)	f'_c (psi)	n	ϕ
1.000	3374.000	3000	0.500	0.65

Results

ϕN_{pn}	=	2193	lb
N_{ua}	=	974	lb
Utilization	=	44.4%	

Table 17.3.1.1



10/18/2024

11.Design Proof Shear Loading

Reference

Steel Strength

ACI 318-14 17.5.1

Variables

V_{sa} (lb)	ϕ
12310.00	0.60

Results

ϕV_{sa}	=	7386	lb
V_{ua}	=	974	lb
Utilization	=	13.2%	

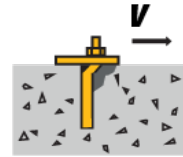


Table 17.3.1.1

Concrete Breakout Strength

ACI 318-14 17.5.2

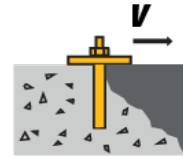
Equations

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

Eqn. 17.5.2.1b

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

Eqn. 17.5.2.2a



Variables

A_{Vc} (in ²)	A_{Vc0} (in ²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
102.000	72.000	1.000	1.000	1.000	1.000
$\Psi_{a,V}$	l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{al} (in)
1.000	2.240	0.625	1.000	3000.000	4.000
V_b (lb)	ϕ				
3130.129	0.70				
		V_{cbg} (lb)			
		4434.349			

Results

ϕV_{cbg}	=	3104	lb
Direction	=	X+	
V_{ua}	=	1948	lb
Utilization	=	62.8%	

Table 17.3.1.1

Shear Breakout Case 2 Controls

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



10/18/2024

Pryout Strength**ACI 318-14 17.5.3****Equations**

$$V_{cpg} = k_{cp} \cdot N_{cpg}$$

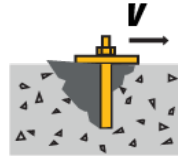
Eqn. 17.5.3.1b

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

Eqn. 17.4.2.1b

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

Eqn. 17.4.2.2a

**Variables**

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
78.758	45.158	1.000	1.000	1.000	1.000
c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
6.300	21.000	1.000	2.24	3000.000	4.000
N_b (lb)	k_{cp}	N_{cpg} (lb)	ϕ		
3856.133	1.000	6725.279	0.70		

Results

ϕV_{cpg}	=	4708.00	lb
V_{ua}	=	1948	lb
Utilization	=	41.4%	

Table 17.3.1.1

12. Interaction of Tension and Shear Loads

Reference

ACI 318-14 17.6

Equations

$$(N_{ua} / \phi \cdot N_n)^{5/3} + (V_{ua} / \phi \cdot V_n)^{5/3} \leq 1.0$$

Eqn. 17.6.3

Variables

$N_{ua} / \phi \cdot N_n$	$V_{ua} / \phi \cdot V_n$
0.446	0.628

Results

0.720	$\leq$	1.0
Status	:	OK

ANCHOR DESIGN CRITERIA IS SATISFIED



Steel Bolts Threaded Tension & Shear Design 14th

Data:

$T_{\text{bolt}} := 1.948$	Tension in Anchor (kips)	$\Omega := 2.00$	ASD Factor=2.00
$V_{\text{bolt}} := 1.948$	Shear in Anchor (kips)		
$F_{\text{nv}} := 45.0$	A325 Nominal Shear Strength (ksi) Table J3.2 AISC 16.1-1200.45*Fu		
$F_{\text{nt}} := 90.0$	A325 Nominal Tensile Strength (ksi) Table J3.2 AISC 16.1-120.....=0.75*Fu		
$D := 0.625$	Screw Diameter (in)	$n := 10.00$	Threads per in..... (AISC-Table 7-17 page 7-81)

$$F_v := \frac{F_{\text{nv}}}{\Omega} \quad F_v = 22.500 \quad \text{ksi}$$

$$F_t := \frac{F_{\text{nt}}}{\Omega} \quad F_t = 45.000 \quad \text{ksi}$$

$$A_{\text{bolt}} := 0.7854 \cdot \left(D - \frac{0.9743}{n} \right)^2 \quad A_{\text{bolt}} = 0.219 \quad \text{in}^2$$

Check Shear

$$f_v := \frac{V_{\text{bolt}}}{A_{\text{bolt}}} \quad f_v = 8.911 \quad \text{ksi}$$

$$V_{\text{allow}} := F_v \cdot A_{\text{bolt}} \quad V_{\text{allow}} = 4.919 \quad \text{kips}$$

$$\text{SHEAR} := \text{if}(f_v \leq F_v, \text{"OK"}, \text{"NG"}) \quad \boxed{\text{SHEAR} = \text{"OK"}}$$

Check Tension

$$f_t := \frac{T_{\text{bolt}}}{A_{\text{bolt}}} \quad f_t = 8.911 \quad \text{ksi}$$

$$F'_{\text{nt}} := \min \left(F_t, \max \left(0, 1.3F_t - \Omega \cdot \frac{F_t}{F_v} \cdot f_v \right) \right) \quad F'_{\text{nt}} = 22.855 \quad \text{ksi}$$

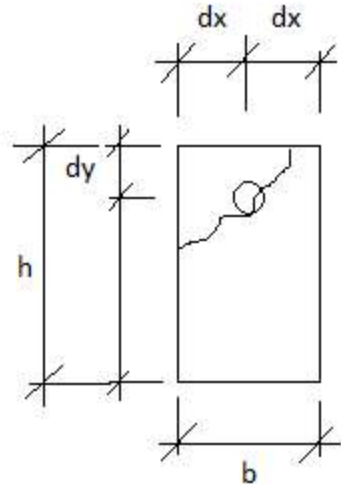
$$T_{\text{allow}} := F'_{\text{nt}} \cdot A_{\text{bolt}} \quad T_{\text{allow}} = 4.996 \quad \text{kips}$$

$$\text{TENSION} := \text{if}(T_{\text{bolt}} \leq T_{\text{allow}}, \text{"OK"}, \text{"NG"}) \quad \boxed{\text{TENSION} = \text{"OK"}}$$

Minimum Thread Length

$$TL_{\text{min}} := \frac{2 \cdot T_{\text{bolt}}}{F_v \cdot \pi \cdot \left(D - \frac{0.9743}{n} \right)} \quad TL_{\text{min}} = 0.104 \quad \text{in}$$

PLATE PL3"x3"x1/4"



General Data:

$D_L := 50.0$ Gravity Load on Connection (lbs)
 $V_{conn} := 1948.0$ Shear on Connection (lbs)
 $T_t := 1948.0$ Tension Load on Connection (lbs)
 $GAP := 1.50$ Maximum dist. from base Plate to load (in)

Anchors Locations Data:

$d_x := 1.5$ Distance from Edge of Plate to Center of Anchor Along XX (in)
 $d_y := 1.5$ Distance from Edge of Plate to Center of Anchor Along YY (in)

Angle Material Data:

$b := 3.0$ Angle Width (in)
 $thk := 0.25$ Angle Thickness (in)
 $h := 3.0$ Angle Length (in)
 $e_v := GAP - thk$ $e_v = 1.25$ in

$d_b := 0.625$ Bolt diam (in)
 $U := 0.750$ Reduction Factor
 $l_c := 0.00$ Spacing (in)
 $\Omega := 2.00$ (Safety factor)

Angle Material Data:

$F_b := 9500.0$ Allowabel Bending Stress (psi)
 $F_v := 5000.00$ Allowabel Shear Stress (psi)
 $F_{ty} := 15000.00$ Allowable Tension Stress for yielding (psi)
 $F_{tf} := 24000.00$ Allowable Tension Stress for fracture (psi)
 $F_y := 15000.00$ Yielding Stress (psi)
 $F_u := 24000.00$ Ultimate Stress (psi)

Shear & Moment per angle:

$$V_{\text{total}} := \frac{D_L}{2} + \frac{V_{\text{conn}}}{2}$$

$$V_{\text{total}} = 999.00 \quad \text{lbs}$$

$$M_V := \frac{(V_{\text{total}} \cdot e_v)}{2}$$

$$M_V = 624.38 \quad \text{lbs} \cdot \text{in}$$

$$S_V := \frac{\text{thk} \cdot (h)^2}{6}$$

$$S_V = 0.38 \quad \text{in}^3$$

$$A_V := b \cdot h$$

$$A_V = 9.00 \quad \text{in}^2$$

$$A_n := \text{thk} \cdot h - \left[3 \cdot \left[\frac{(3.1415 \cdot d_b^2)}{4} \right] \right]$$

$$A_n = -0.17 \quad \text{in}^2$$

Check Bending Stress :

$$fb_V := \frac{M_V}{S_V}$$

$$fb_V = 1665.00 \quad \text{psi}$$

$$\text{BENDING}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } F_b \geq fb_V \end{cases}$$

$$\text{BENDING}_V = \text{"OK"}$$

Check Tension Stress :

$$f_{ty} := \frac{(T_t)}{0.6 \cdot A_V} \quad \text{for yielding} \quad f_{ty} = 360.7 \text{ psi}$$

$$f_{tf} := \frac{(T_t)}{0.5 \cdot (U \cdot A_n)} \quad \text{for fracture} \quad f_{tf} = -3049 \text{ psi}$$

$$\text{TENSION}_{\text{yielding}} := \text{if}(F_{ty} \geq f_{ty}, \text{"OK"}, \text{"NG"})$$

$$\text{TENSION}_{\text{yielding}} = \text{"OK"}$$

$$\text{TENSION}_{\text{fracture}} := \text{if}(F_{tf} \geq f_{tf}, \text{"OK"}, \text{"NG"})$$

$$\text{TENSION}_{\text{fracture}} = \text{"OK"}$$

Check Shear Stress :

$$fv_V := \frac{V_{\text{total}}}{A_V}$$

$$fv_V = 111.00 \quad \text{psi}$$

$$\text{SHEAR}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } F_v \geq fv_V \end{cases}$$

$$\text{SHEAR}_V = \text{"OK"}$$

Check Combined Bending & Shear :

$$IE_V := \frac{fb_V}{F_b} + \frac{fv_V}{F_v} + \max\left(\frac{f_{ty}}{F_{ty}}, \frac{f_{tf}}{F_{tf}}\right)$$

$$IE_V = 0.22$$

$$\text{COMBINED}_V := \begin{cases} \text{"N.G."} \\ \text{"OK"} & \text{if } 1 \geq IE_V \end{cases}$$

$$\text{COMBINED}_V = \text{"OK"}$$

Bearing Strength on angle:

$$R_n := \left[\frac{(2.0 \cdot d_b \cdot \text{thk} \cdot F_u)}{\Omega} \right]$$

Forbs slotted holes perp
to direction of force

$$R_n = 3750.00 \quad \text{lbs}$$

$$\text{Bearing}_{\text{strength}} := \text{if}\left[R_n \geq (V_{\text{total}}), \text{"OK"}, \text{"NG"}\right]$$

$$\text{Bearing}_{\text{strength}} = \text{"OK"}$$

Check Horizontal Plate Stress

Check at Plane of Failure for Moment :

$$b_{\text{plane}} := b$$

$$b_{\text{plane}} = 3.00 \quad \text{in}$$

$$d_{\text{plane}} := \frac{h}{2} - 1.5$$

$$d_{\text{plane}} = 0.00 \quad \text{in}$$

$$S_h := \frac{b_{\text{plane}} \cdot h^2}{6}$$

$$S_v = 0.38 \quad \text{in}^3$$

Check Bending Stress :

$$fb_h := \frac{M_v}{S_h}$$

$$fb_h = 138.75 \quad \text{psi}$$

$$\text{BENDING}_h := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_b \geq fb_h \end{cases}$$

$$\boxed{\text{BENDING}_h = \text{"OK"}}$$

Check at Plane of Failure for Moment :

$$V_h := V_{\text{total}}$$

$$V_h = 999.00 \quad \text{lbs}$$

$$A_h := b \cdot h$$

$$A_h = 9.00 \quad \text{in}^2$$

Check Shear Stress :

$$fv_h := \frac{V_h}{A_h}$$

$$fb_h = 138.75 \quad \text{psi}$$

$$\text{SHEAR}_h := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq fv_h \end{cases}$$

$$\boxed{\text{SHEAR}_h = \text{"OK"}}$$

Alum. Weld Design For Flat bar 3x3x1/4 to Tube 6x2

Weld and Section Data:

$F_v := 21.00$ Allowable Stress for Welding (ksi)

Welding: Alum. Filler Alloy 5356

$b := 0.25$ Width of Tube Alum. (in)

**Allowable Stress for Welding:
 $F_v = 7.00$ Ksi**

$d := 3.0$ Depth of Tube Alum. (in)

Load Data:

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

$V_{weld} := 1.948$ Shear (Kips)

$P_{weld} := 0.0$ Tension or Compression (Kips)

$T_{weld} := 0.00$ Torsional Moment (kips-ft)

Check Weld Section :

$$t_e := \begin{cases} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\frac{P_{weld}}{2(b+d) \cdot x} + \frac{M_{weld} \cdot 12}{\left(b \cdot d + \frac{d^2}{3}\right) \cdot x} \right]^2 + \left[\frac{1.5V_{weld}}{2(b+d) \cdot x} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6} \cdot x} \right]^2} \geq F_v \\ x \leftarrow x + 0.01 \end{cases} \quad t_e = 0.03 \text{ in}$$

$$S_{weld} := \left(b \cdot d + \frac{d^2}{3}\right) \cdot t_e \quad S_{weld} = 0.11 \text{ in}^3$$

$$A_{weld} := 2 \cdot (d + b) \cdot t_e \quad A_{weld} = 0.20 \text{ in}^2$$

$$J_{weld} := \frac{(b+d)^3}{6} \cdot t_e \quad J_{weld} = 0.17 \text{ in}^3$$

Check Bending Stress :

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

Check Shear Stress :

$$f_v := \frac{1.5V_{\text{weld}}}{A_{\text{weld}}} + \frac{T_{\text{weld}} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{\text{weld}}} \quad f_v = 14.98 \quad \text{ksi}$$

Check Tension Stress :

$$f_a := \frac{P_{\text{weld}}}{A_{\text{weld}}} \quad f_a = 0.00 \quad \text{Ksi}$$

Check Combined Stress :

$$f_{\text{weld}} := \sqrt{(f_b + f_a)^2 + f_v^2} \quad f_{\text{weld}} = 14.98 \quad \text{ksi}$$

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

STRESS = "OK"

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

W_{weld} = "3/16"