STRUCTURAL CALCULATIONS

Rooftop Screen Rail Design

Prepared For:

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Prepared By:

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RISE Project No. 24075

April 4th, 2024

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of South Dakota.

Justin Chistensen	**South Dakota License Number**	**Date**

LICENSED PROFESSIONAL ENGINEER
REG. No
11152
JUSTIN
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SOUTH DAKOTA

04/04/2024

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RISE STRUCTURAL ASSOCIATES INC.
3285 Fiechtner Dr. S. • Ste. B
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Phone: (701) 318-1782
www.riseincorp.com

DATE 4/1/24 NO. 24075

PROJECT Rooftop Screen Wall Rail Desin

SUBJECT

Wind = V = 120 mph
Risk Category = II
Exposure = C

SEE MECAWIND → F = 756.16 / (8' × 5')
= 18.9 psf

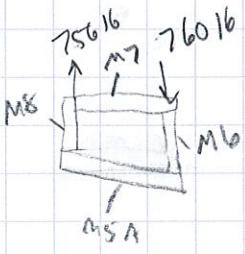
Post Height = 8'-0" spaced 5'-0" Apart

W_w = 18.9 psf × 5' = 94.5 plf

SEE RISA : HSS 4×4 × 1/4" OK
HSS 2×2 × 1/8" Kicker OK

SEE RISA : 4×4 × 1/4" Aluminum 6063-T52 OK
2×2 × 1/8" Aluminum 6063-T52 Kicker OK

R_1 = Uplift = 756 lb Down = 766 lb
Shear = 753 lb
R_2 = Uplift = 756 lb = 640 lb Down = 760 lb
Shear = 750 lb



Channels = 8" × 2.29" × .25" 6061-T6 AS

SEE RISA CALCS

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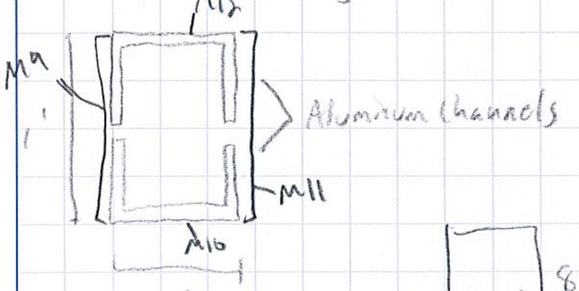
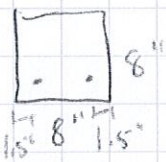
DATE 4/3/24 NO. 24075
PROJECT Roof-top Screen Rail Design
SUBJECT

Pedestal Desing

SEE RISA CALCS



Use 8" x 2.29" x .25" Channels
6061 T6 - AS

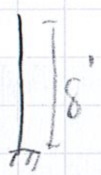


Tension = 3629 lb

Compression = 3629 lb

1,512 lb·ft x 12 = 18,144 . / 5 = 3629 lb

Base Plate Design



$W_W = 94.5$ psf $\times \frac{8}{2}$ = 378 x 4 = 1,512 lb·ft
Moment about Base = 1.51 Kip·ft

Use 8x8x.75" or 8" x 10" x .75"

Kicker Mounting Bracket

Tensile Strength = $P_n = \frac{F_y A_g}{1.67} = \frac{(36\text{ ksi})(3/8" \times 10")}{1.67}$

Plate Attachment to Post

= 80.84 kips > 700 lb

Use 1/4" Fillet Weld

Plate OK STEEL

1/4" Fillet Weld for Steel = 3710 lb/in
Weld Length = 4" x 3710 = 14840 lb > 700
2" Weld on Top + Bott.
Weld OK

$= \frac{(35\text{ ksi})(3/8 \times 10)}{1.67}$ → check Red. Factors

= 78.59 kips > 760 lb

1/4" Fillet Weld for Aluminum

PLATE OK ALUM

$R_n = F_{sw}$ .707(.25) = 15ksi (.707)(.25)
= 2.65125 kip·in

2651 lb/in x 4" = 10,605 lb > 760 lb

Bolt Connection between plate + kicker

5/8" Bolt UNC x 3-3/4" Hex-head cap screw (A4 Steel)

Stainless steel Bolt Shear strength = Group B
Use $F_{nv} = 68$ ksi

$R_n = \frac{F_n A_b}{2} = \frac{68\left(\frac{5/8^2 \pi}{4}\right)}{2} = 10.43$ kips

10,431 lb > 700 lb
Bolt is OK

Bolt is OK for plate connection upon inspection

Channel welds

2559 lb/in

Weld strength = 3629 lb / 2651 lb/in = 1.37 in" Use 2" of weld @ Minimum for Channel to Channel Connections

MecaWind v2405


```
Calculations Prepared by:                    Calculations Prepared For:
Client:          0
Date: Apr 04, 2024                            Project #:         0
Designer: 0                                   Location:          0

File Location : C:\Users\kmswo\Dropbox\24075_ Rooftop Screen Rail Design\Wind.wnd
```

Basic Wind Parameters

```
Wind Load Standard              = ASCE 7-16  Exposure Category              = C
Wind Design Speed               = 120.0 mph  Risk Category                  = II
Structure Type                  = Other      Other Structure Type           = Freestanding Wall
```

General Wind Settings

```
Incl_LF    = Include ASD Load Factor of 0.6 in Pressures                 = True
DynType    = Dynamic Type of Structure                                   = Rigid
Zg         = Altitude (Ground Elevation) above Sea Level                 = 0.000 ft
Bdist      = Base Elevation of Structure                                 = 0.000 ft
Reacs      = Show the Base Reactions in the output                       = False
MWFRSType = MWFRS Method Selected                                        = Ch 29
```

Topographic Factor per Fig 26.8-1

```
Topo       = Topographic Feature                                         = None
Kzt        = Topographic Factor                                          = 1.000
```

Freestanding Wall Inputs

```
h     : Height to Top of Wall      = 8.000 ft   B     : Horizontal Width of Wall   = 5.000 ft
Lr    : Dimension of return corner = 5.000 ft   s     : Vertical Height of Wall    = 8.000 ft
e     : Solidity Ratio             = 0.800      t     : Thickness of Wall          = 0.333 ft
Dbl   : Double Faced & all sides enclosed= False  IsCol: Is the Wall Supported on Columns= False
```

Exposure Constants per Table 26.11-1:

```
Alpha: Table 26.11-1 Const         = 9.500      Zg:    Table 26.11-1 Const        = 900.000 ft
At:    Table 26.11-1 Const         = 0.105      Bt:    Table 26.11-1 Const        = 1.000
Am:    Table 26.11-1 Const         = 0.154      Bm:    Table 26.11-1 Const        = 0.650
C:     Table 26.11-1 Const         = 0.200      Eps:   Table 26.11-1 Const        = 0.200
```

Gust Factor Calculation:

```
Gust Factor Category I Rigid Structures - Simplified Method
G1         = For Rigid Structures (Nat. Freq.>1 Hz) use 0.85             = 0.85
Gust Factor Category II Rigid Structures - Complete Analysis
Zm         = Max(0.6 * Ht, Zmin)                                         = 15.000 ft
Izm        = Cc * (33 / Zm) ^ 0.167                                      = 0.228
Lzm        = L * (Zm / 33) ^ Eps                                         = 427.057
B          = Structure Width Normal to Wind                              = 5.000 ft
Q          = (1 / (1 + 0.63 * ((B + Ht) / Lzm)^0.63))^0.5                = 0.967
G2         = 0.925*((1+0.7*Izm*3.4*Q)/(1+0.7*3.4*Izm))                   = 0.908
Gust Factor Used in Analysis
G          = Lessor Of G1 Or G2                                          = 0.850
```

Main Wind Force Resisting System (MWFRS) Calculations for Freestanding Wall per Ch 29:

```
LF         = Load Factor based upon ASD Design                           = 0.60
hs         = Overall height of structure                                 = 8.000 ft
h          = Mean Roof Height above grade                                = 8.000 ft
Kh         = Z < 15 ft [4.572 m]--> (2.01 * (15/zg)^(2/Alpha) {Table 26.10-1}= 0.849
Kzt        = Topographic Factor is 1 since no Topographic feature specified  = 1.000
Kd         = Wind Directionality Factor per Table 26.6-1                 = 0.85
qh         = (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF                   = 15.96 psf
```

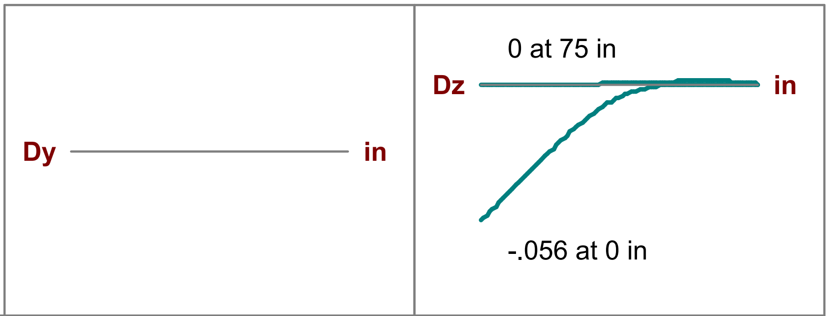
MWFRS Pressures on Freestanding Wall per Fig 29.3-1:

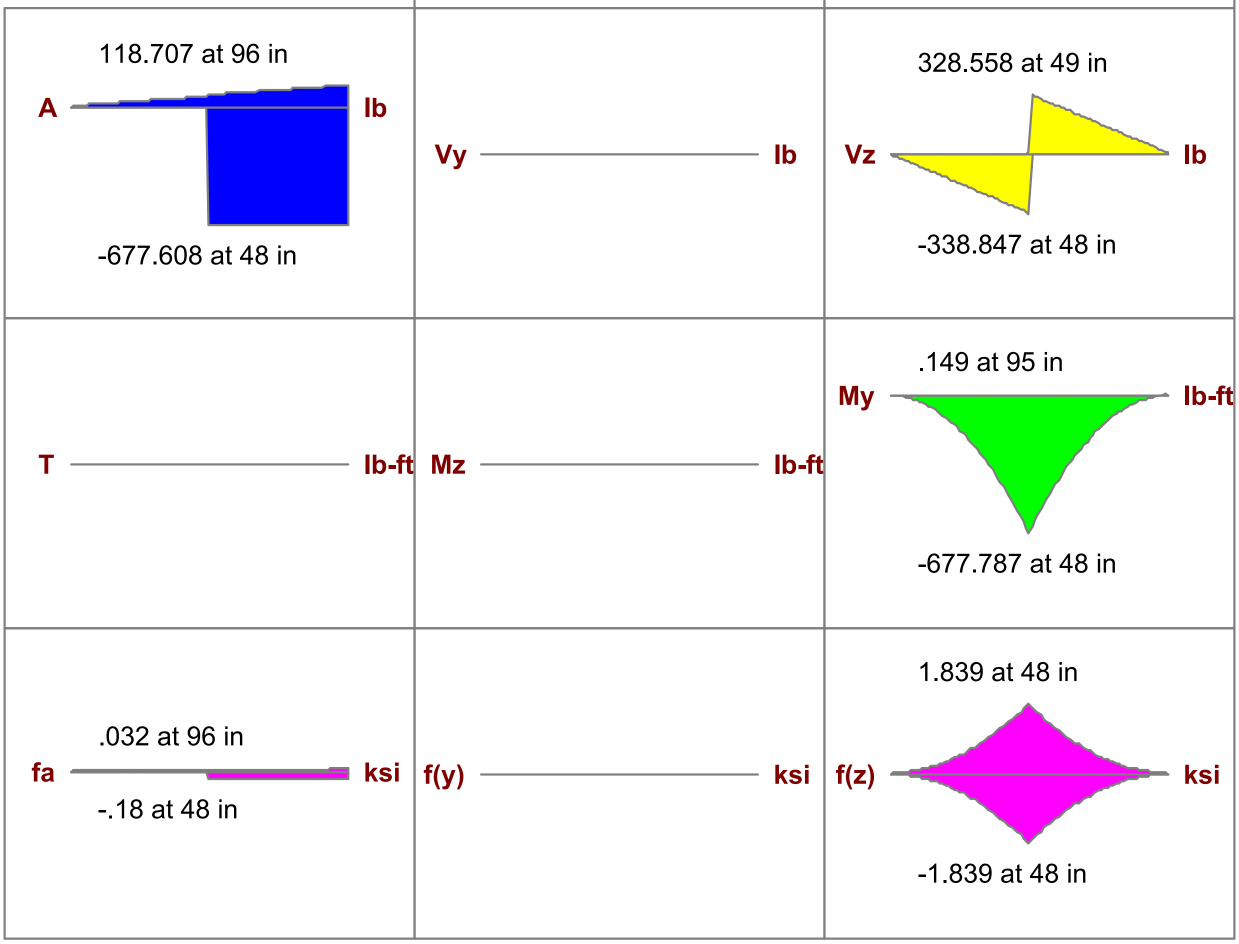
```
R          = Reduction factor to account for openings:  (1-(1-e)^1.5)    = 0.911
Rc         = Reduction factor for Case C since s/h > 0.8: (1.8-s/h)      = 0.800
As         = Gross Area of Wall:  B * s                                  = 40.00 sq ft
B/s        = Aspect Ratio:  B / s                                        = 0.625
s/h        = Clearance Ratio: s / h                                      = 1.000
Cf         = Net Force Coefficient for Case A and B per Fig 29.3-1       = 1.530
e          = Not Double Faced, Case B eccentricity is 0.2                = 0.2

Case A:  Resultant force acts normal to face through geometric center
         and since s/h = 1 then consider force acting 0.05*s above the geometric center

0.05*s     = Since s/h=1, load applied at vertical offset from geom center  = 0.400 ft
F          = Design Wind force:  qh * G * Cf * As * R                    = 756 lb
```

```
Case B:  Resultant force acts normal to face at a distance from the geometric
         center toward the windward edge equal to e times the average width
         and since s/h = 1 then consider force acting 0.05*s above the geometric center

0.05*s    = Since s/h=1, load applied at vertical offset from geom center   = 0.400 ft
Dx        = Force Offset from Center toward windward edge:  e * B           = 1.000 ft
F         = Design Wind force:  qh * G * Cf * As * R                        = 756 lb

Case C:  Since B/s < 2 then Case C need not be considered
```

Member: ***M2***

Shape: **HSS4X4X4_A1085**
Material: **A500 Gr.B Rect**
Length: **96 in**
I Joint: **N4**
J Joint: **N3**
Envelope
Code Check: **0.059 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.059 (LC 9)**	Max Shear Check	**0.013 (z) (LC 5)**
Location	**48 in**	Location	**48 in**
Equation	**H1-1b**	Max Defl Ratio	**L/10000**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**96 in**	**96 in**
Pnc/om	**79532.989 lb**	KL/r	**62.576**	**62.576**
Pnt/om	**103500.85 lb**			
Mny/om	**12145.315 lb-ft**	L Comp Flange	**96 in**	
Mnz/om	**12145.315 lb-ft**	L-torque	**96 in**	
Vny/om	**26900.716 lb**	Tau_b	**1**	
Vnz/om	**26900.716 lb**			
Tn/om	**9618.149 lb-ft**			
Cb	**1.572**			

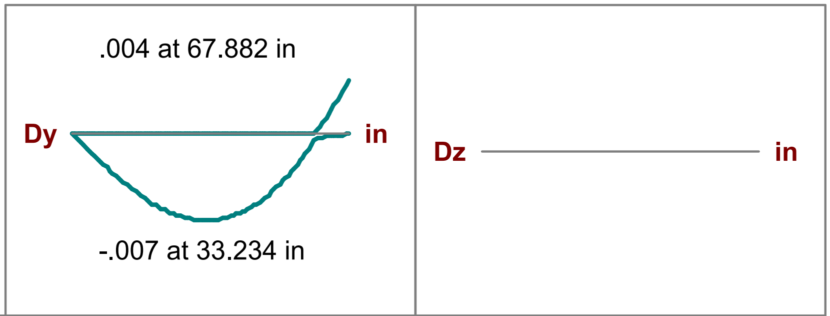
Member: ***M3***

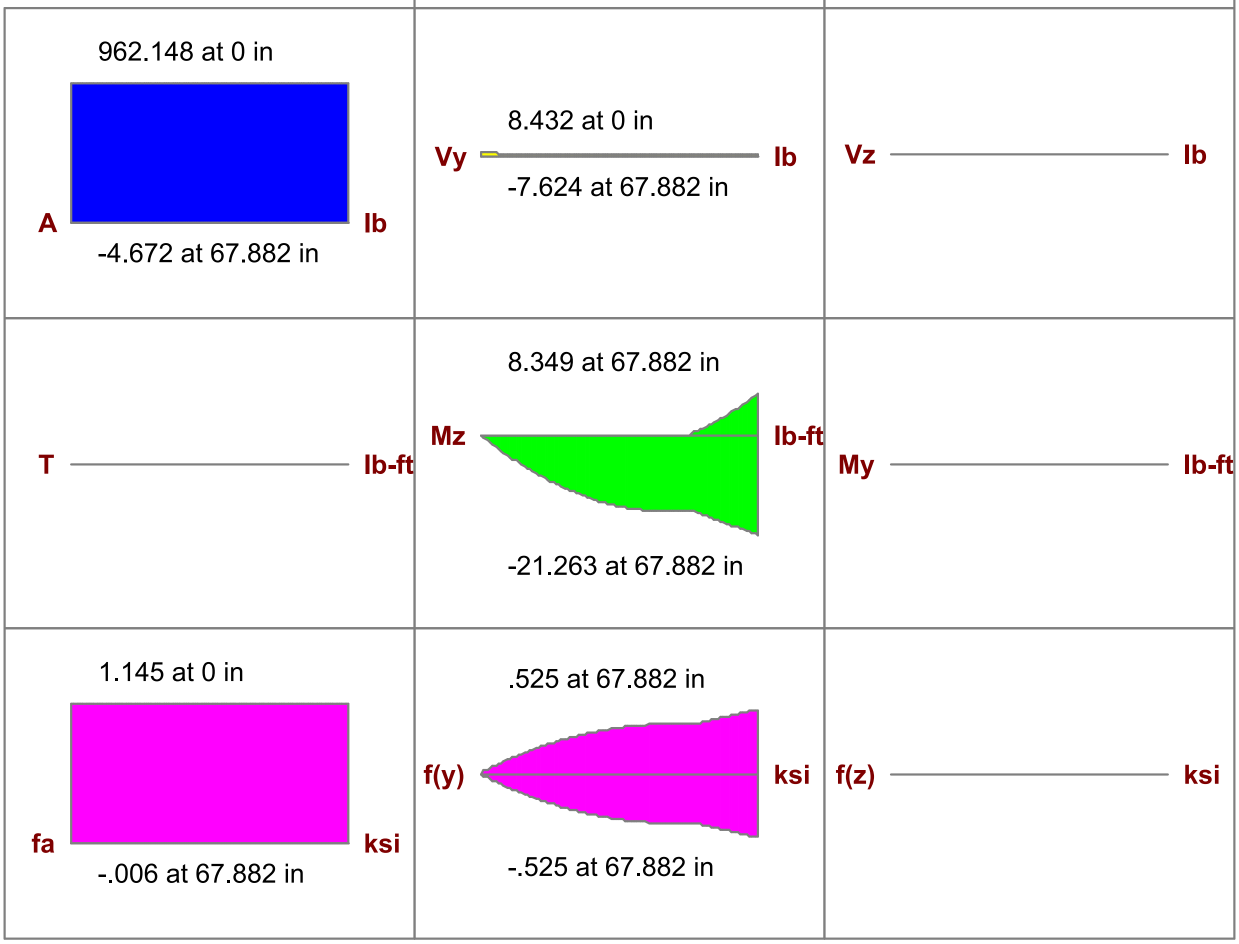
Shape:	**HSS2X2X2**
Material:	**A500 Gr.B Rect**
Length:	**67.882 in**
I Joint:	**N5**
J Joint:	**N6**

Envelope

Code Check: **0.071 (LC 5)**

Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

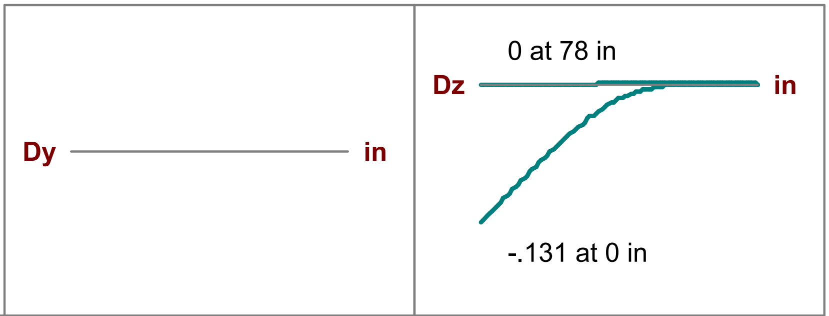
Max Bending Check	**0.071 (LC 5)**	Max Shear Check	**0.001 (y) (LC 5)**
Location	**0 in**	Location	**0 in**
Equation	**H1-1b***	Max Defl Ratio	**L/8035**

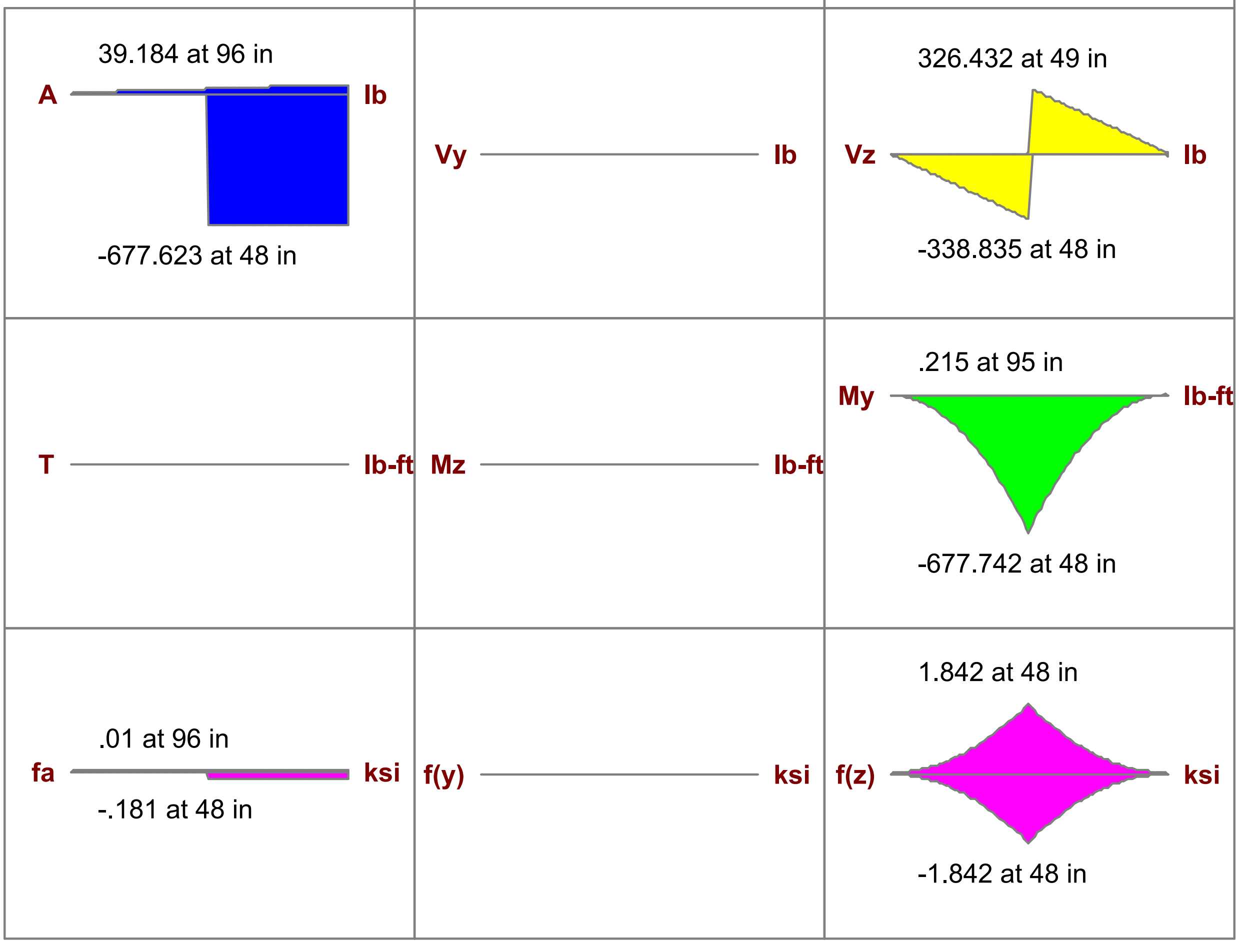
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**67.882 in**	**67.882 in**
Pnc/om	**13540.877 lb**	KL/r	**89.244**	**89.244**
Pnt/om	**23137.725 lb**			
Mny/om	**1340.519 lb-ft**	L Comp Flange	**67.882 in**	
Mnz/om	**1340.519 lb-ft**	L-torque	**67.882 in**	
Vny/om	**6334.183 lb**	Tau_b	**1**	
Vnz/om	**6334.183 lb**			
Tn/om	**1125.822 lb-ft**			
Cb	**1.14**			

Member: ***M4***

Shape: **RT4X4X0.250**
Material: **6063-T5**
Length: **96 in**
I Joint: **N8**
J Joint: **N7**
Envelope
Code Check: **0.209 (LC 9)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.209 (LC 9)**	Max Shear Check	**0.036 (z) (LC 5)**
Location	**48 in**	Location	**48 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

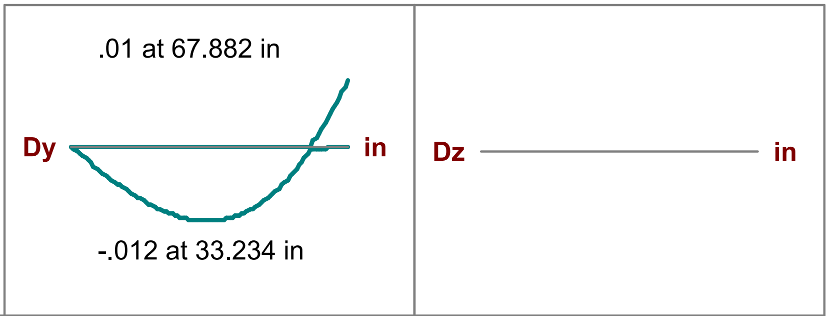
		Slender. Limit λ1	λ2	Slender. Ratio λ	Gov Eqn
Pnt/om	**36363.636 lb**				**D.2-1**
Pnc/om	**26812.641 lb**		**98.9**	**62.6**	**E.2-1**
Mny/om	**3567.677 lb-ft**				**B.5.4.2**
Mnz/om	**3567.677 lb-ft**				**B.5.4.2**
Vny/om	**9454.545 lb**	**0**	**0**	**14**	**G.1-1**
Vnz/om	**9454.545 lb**	**43.6**	**96**	**14**	**G.1-1**

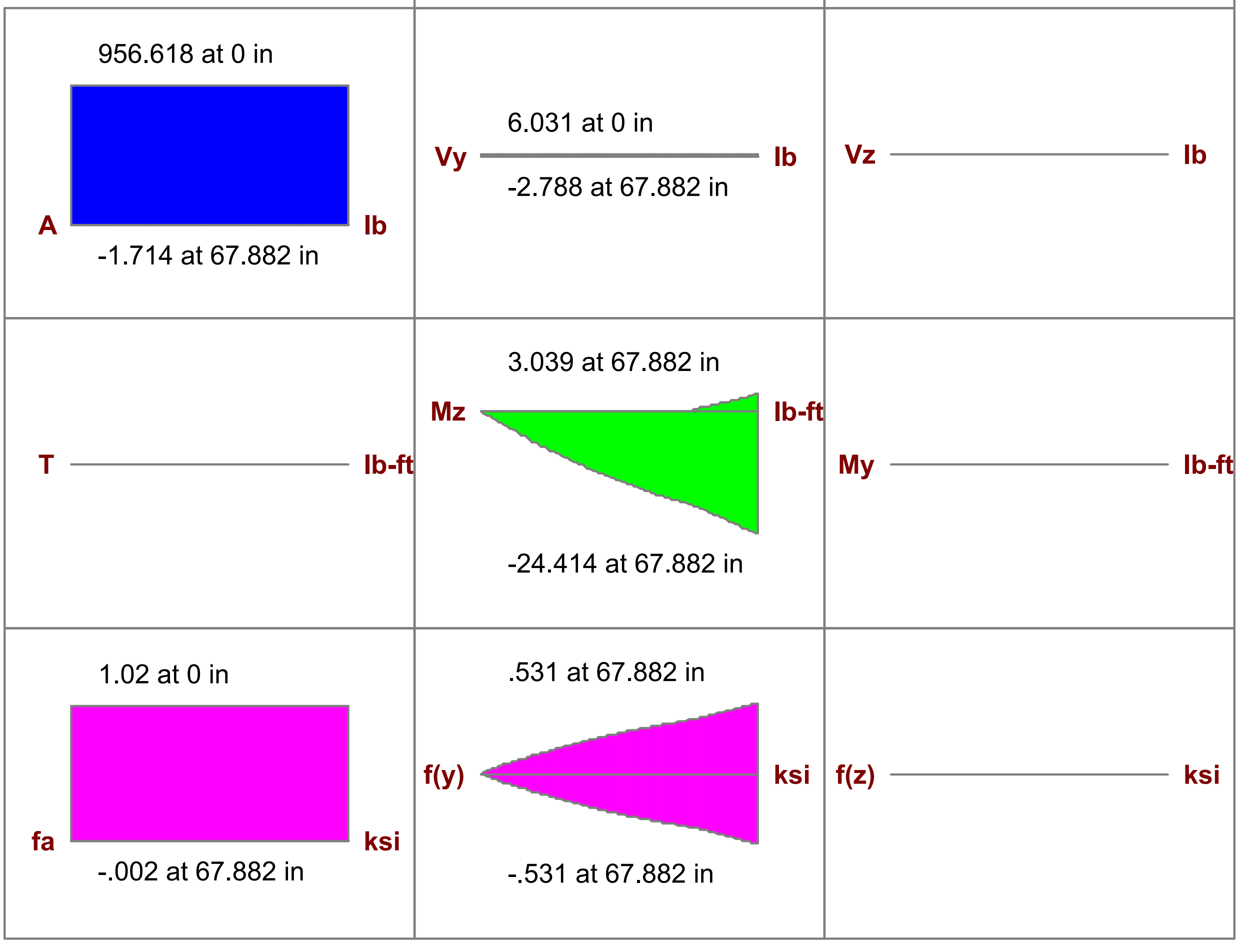
	y-y	z-z
Lb	**96 in**	**96 in**
KL/r	**62.561**	**62.561**

L Comp Top	**96 in**
L Comp Bot	**96 in**
Torque Length	**96 in**
Tau_b	**1**
Cb	**1**

Member: ***M5***

Shape: **RT2X2X0.125**
Material: **6063-T5**
Length: **67.882 in**
I Joint: **N9**
J Joint: **N10**
Envelope
Code Check: **0.230 (LC 9)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.230 (LC 9)**	Max Shear Check	**0.003 (y) (LC 5)**
Location	**67.882 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/4036**

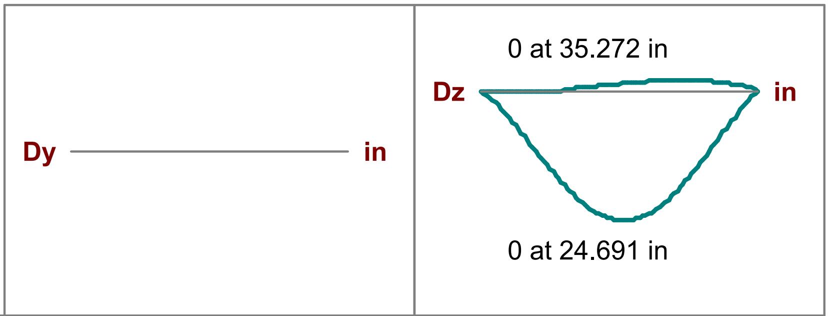
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**9095.758 lb**				**D.2-1**
Pnc/om	**5430.747 lb**		**98.9**	**88.5**	**E.2-1**
Mny/om	**446.061 lb-ft**				**B.5.4.2**
Mnz/om	**446.061 lb-ft**				**B.5.4.2**
Vny/om	**2363.636 lb**	**0**	**0**	**14**	**G.1-1**
Vnz/om	**2363.636 lb**	**43.6**	**96**	**14**	**G.1-1**

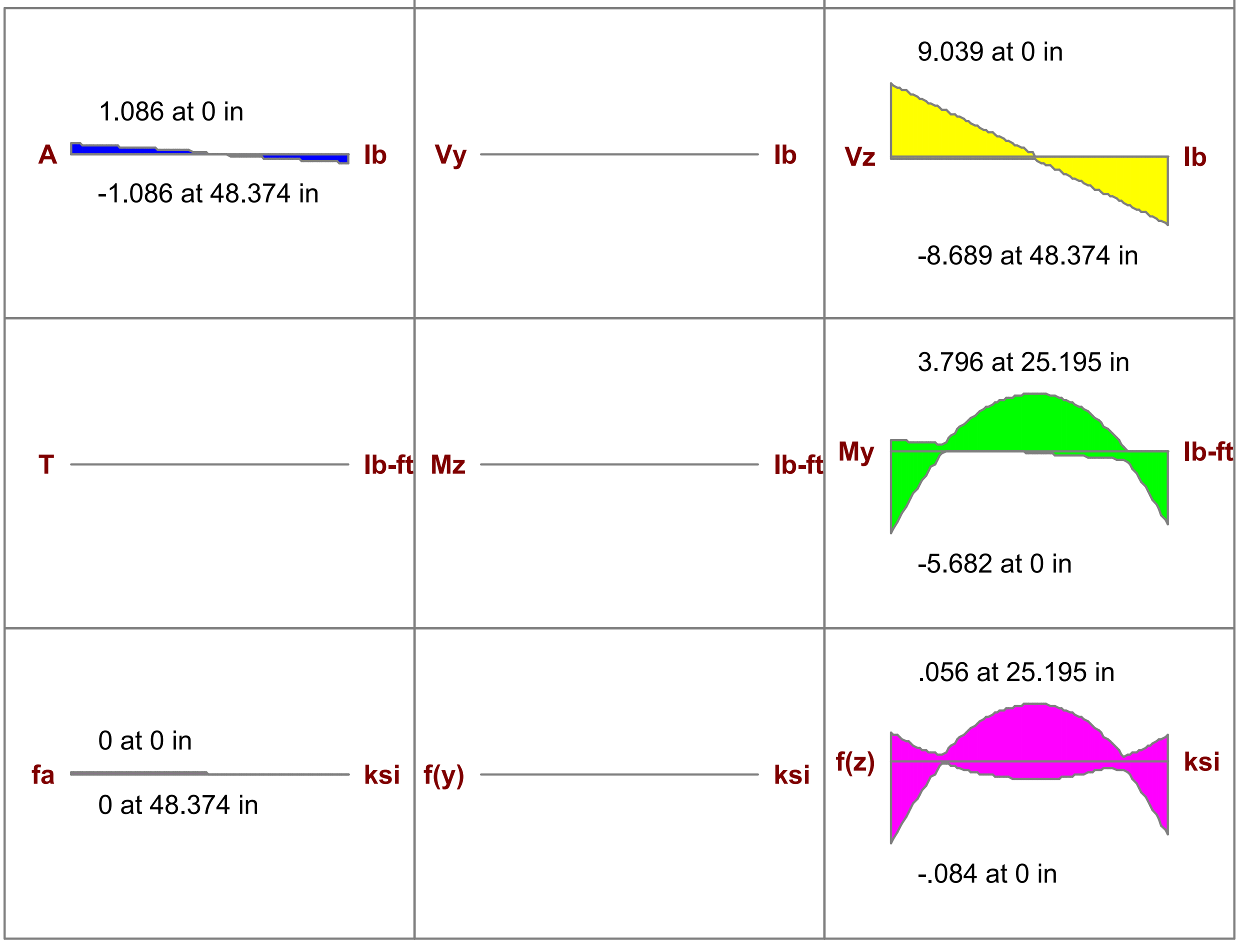
	y-y	z-z
Lb	**67.882 in**	**67.882 in**
KL/r	**88.489**	**88.489**

L Comp Top	**67.882 in**
L Comp Bot	**67.882 in**
Torque Length	**67.882 in**
Tau_b	**1**
Cb	**1.667**

Member: ***M5A***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **48.374 in**
I Joint: **N11**
J Joint: **N9A**
Envelope
Code Check: **0.006 (LC 4)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

-- Pu was ignored in the calculation of the unity check --

Max Bending Check	**0.006 (LC 4)**	Max Shear Check	**0.001 (z) (LC 4)**
Location	**0 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

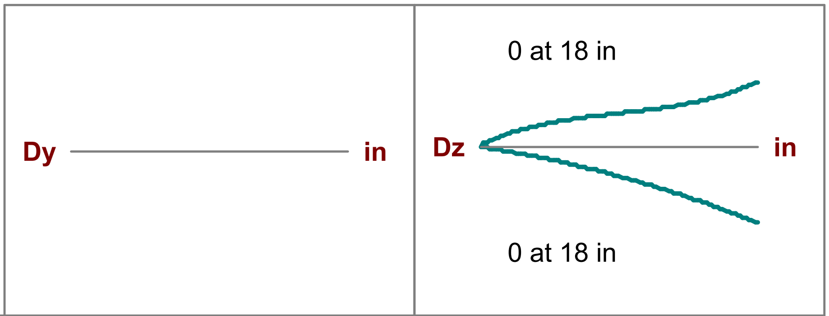
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**20871.775 lb**		**133.3**	**77.8**	**E.2-1**
Mny/om	**919.601 lb-ft**				**B.5.4.2**
Mnz/om	**6229.803 lb-ft**				**B.5.4.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

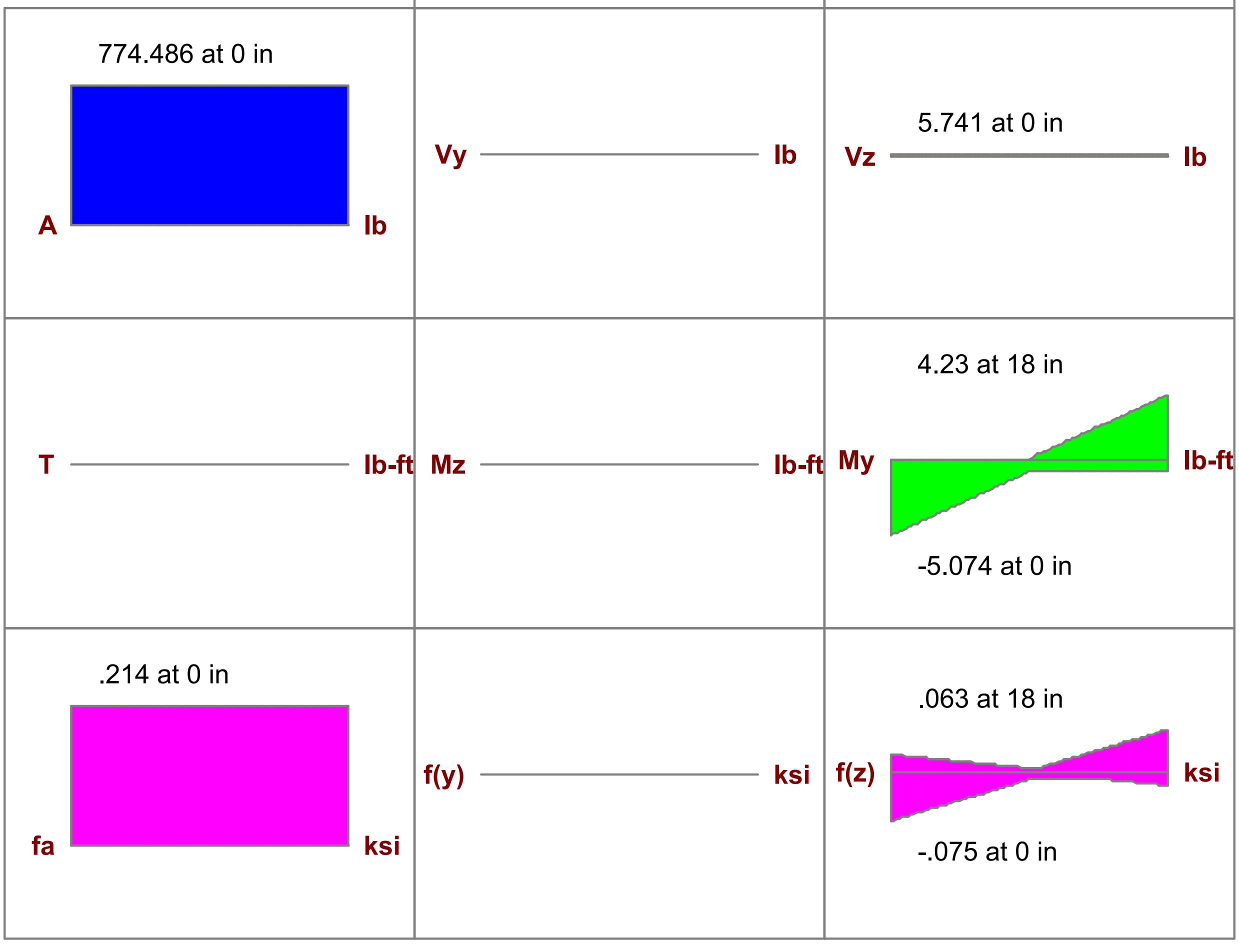
	y-y	z-z
Lb	**48.374 in**	**48.374 in**
KL/r	**77.785**	**15.807**

L Comp Top	**48.374 in**
L Comp Bot	**48.374 in**
Torque Length	**48.374 in**
Tau_b	**1**
Cb	**1**

Member: ***M6***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **18 in**
I Joint: **N9A**
J Joint: **N10A**
Envelope
Code Check: **0.030 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.030 (LC 5)**	Max Shear Check	**0.001 (z) (LC 5)**
Location	**0 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

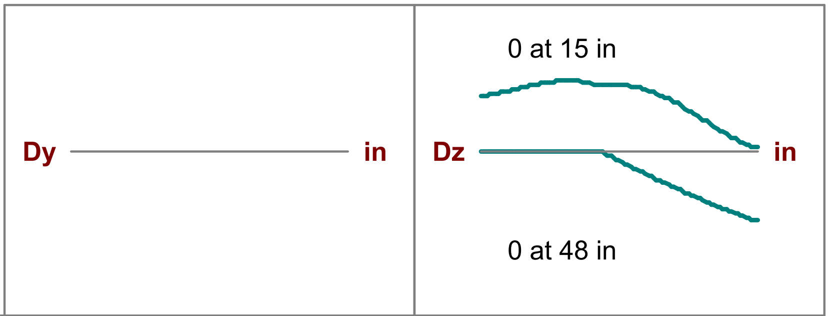
		Slender. Limit $\lambda1$	$\lambda2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**31290.252 lb**		**133.3**	**28.9**	**E.2-1**
Mny/om	**919.601 lb-ft**				**B.5.4.2**
Mnz/om	**6420.455 lb-ft**				**B.5.4.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

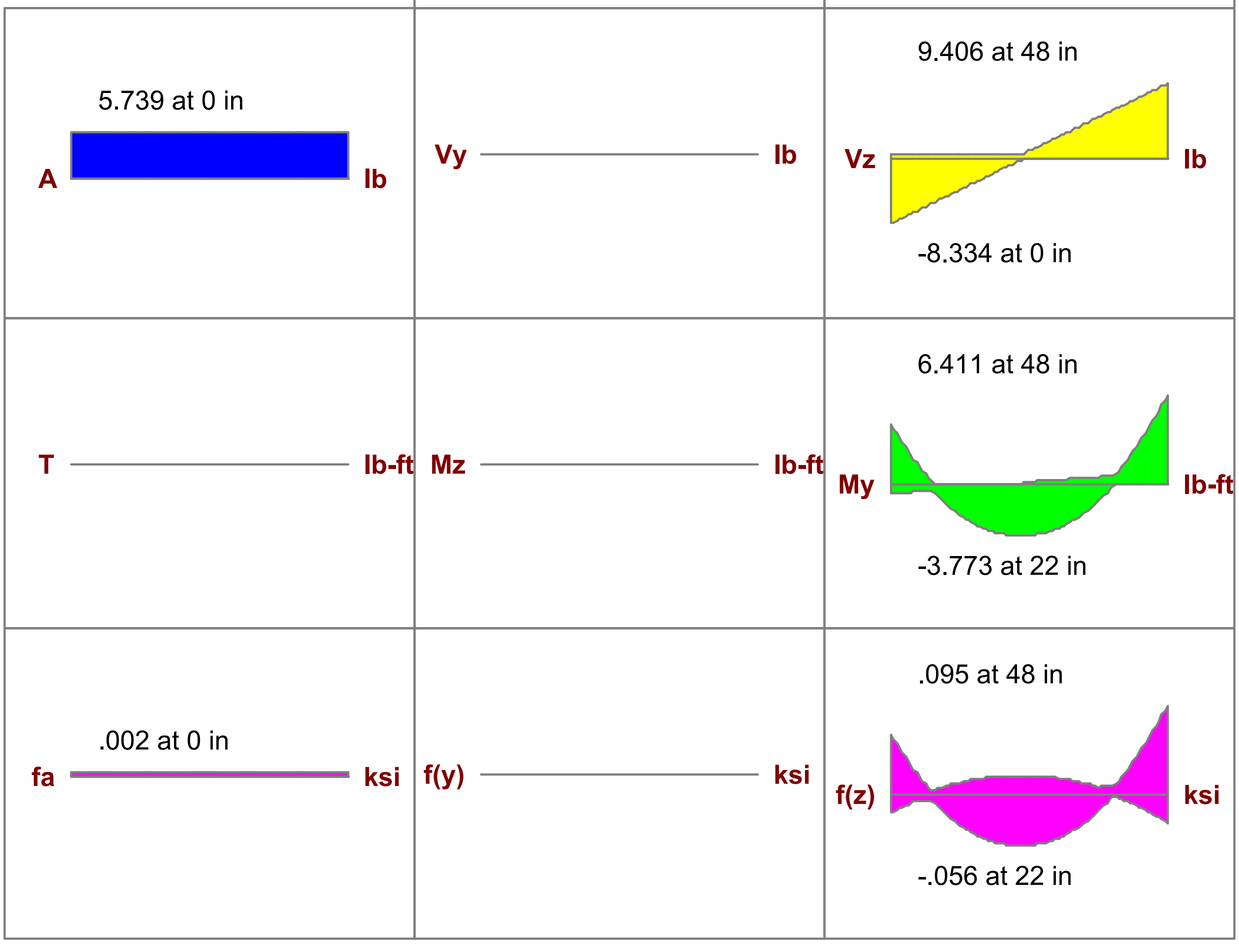
	y-y	z-z
Lb	**18 in**	**18 in**
KL/r	**28.944**	**5.882**

L Comp Top	**18 in**
L Comp Bot	**18 in**
Torque Length	**18 in**
Tau_b	**1**
Cb	**1**

Member: ***M7***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **48 in**
I Joint: **N10A**
J Joint: **N12**
Envelope
Code Check: **0.007 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

-- Pu was ignored in the calculation of the unity check --

Max Bending Check	**0.007 (LC 5)**	Max Shear Check	**0.001 (z) (LC 5)**
Location	**48 in**	Location	**48 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

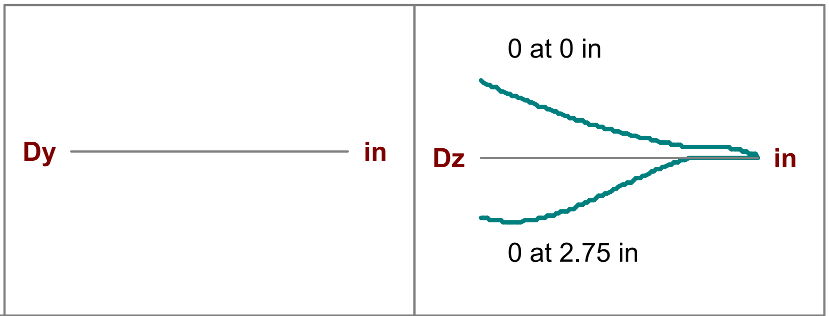
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**20992.705 lb**		**133.3**	**77.2**	**E.2-1**
Mny/om	**919.601 lb-ft**				**B.5.5.2**
Mnz/om	**6240.203 lb-ft**				**B.5.5.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

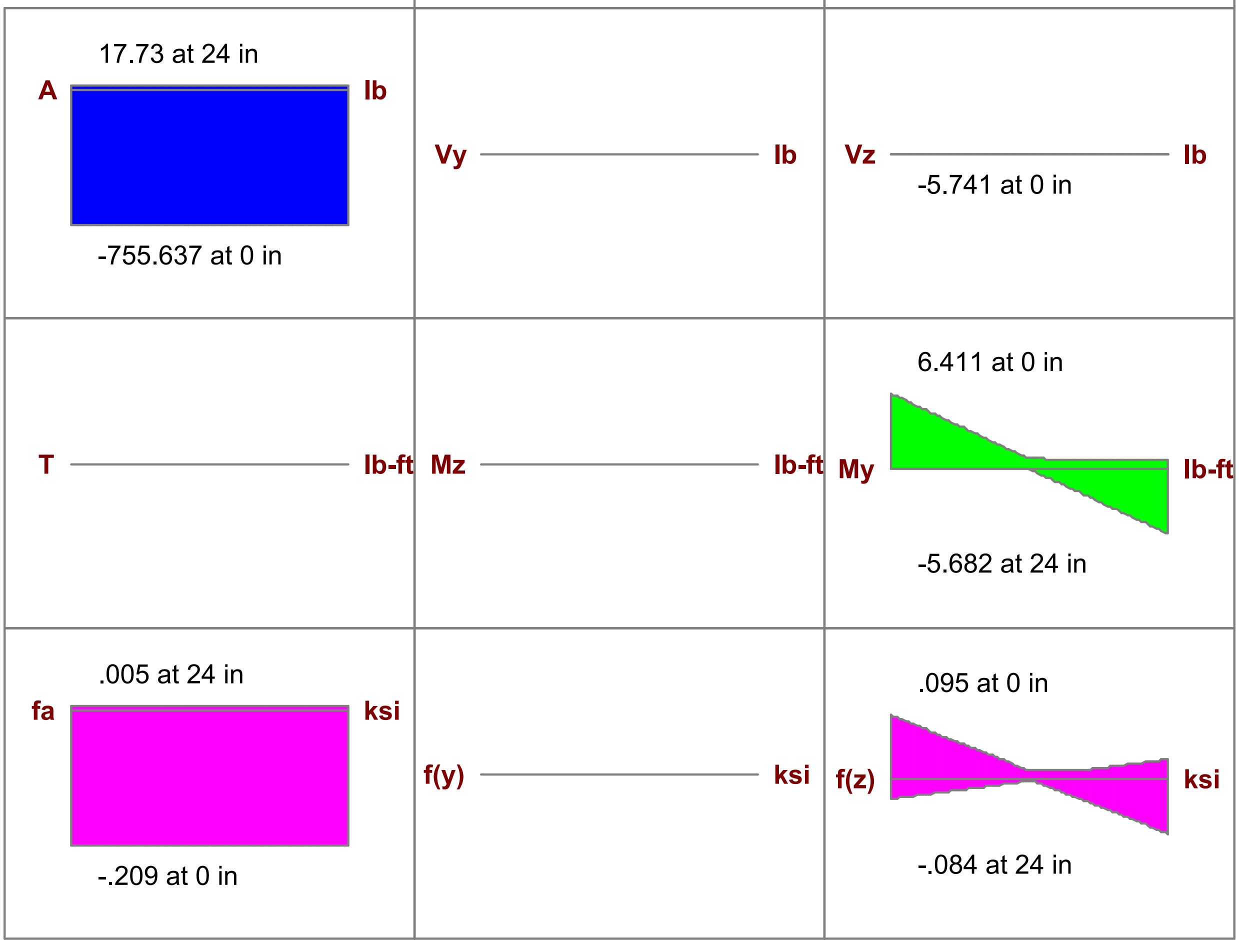
	y-y	z-z
Lb	**48 in**	**48 in**
KL/r	**77.185**	**15.685**

L Comp Top	**48 in**
L Comp Bot	**48 in**
Torque Length	**48 in**
Tau_b	**1**
Cb	**1**

Member: ***M8***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **24 in**
I Joint: **N12**
J Joint: **N11**
Envelope
Code Check: **0.030 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.030 (LC 5)**	Max Shear Check	**0.001 (z) (LC 5)**
Location	**0 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

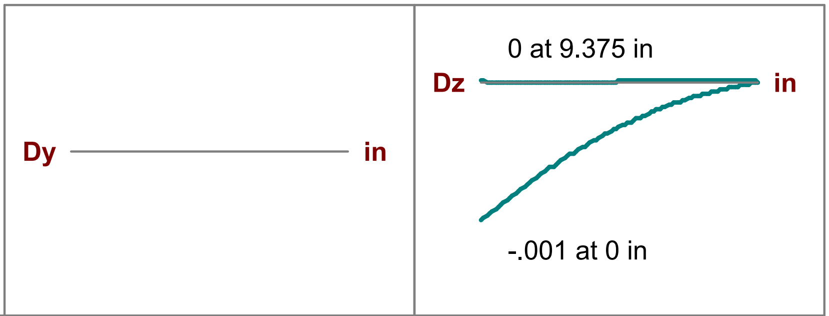
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**29138.214 lb**		**133.3**	**38.6**	**E.2-1**
Mny/om	**919.601 lb-ft**				**B.5.5.2**
Mnz/om	**6420.455 lb-ft**				**B.5.5.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

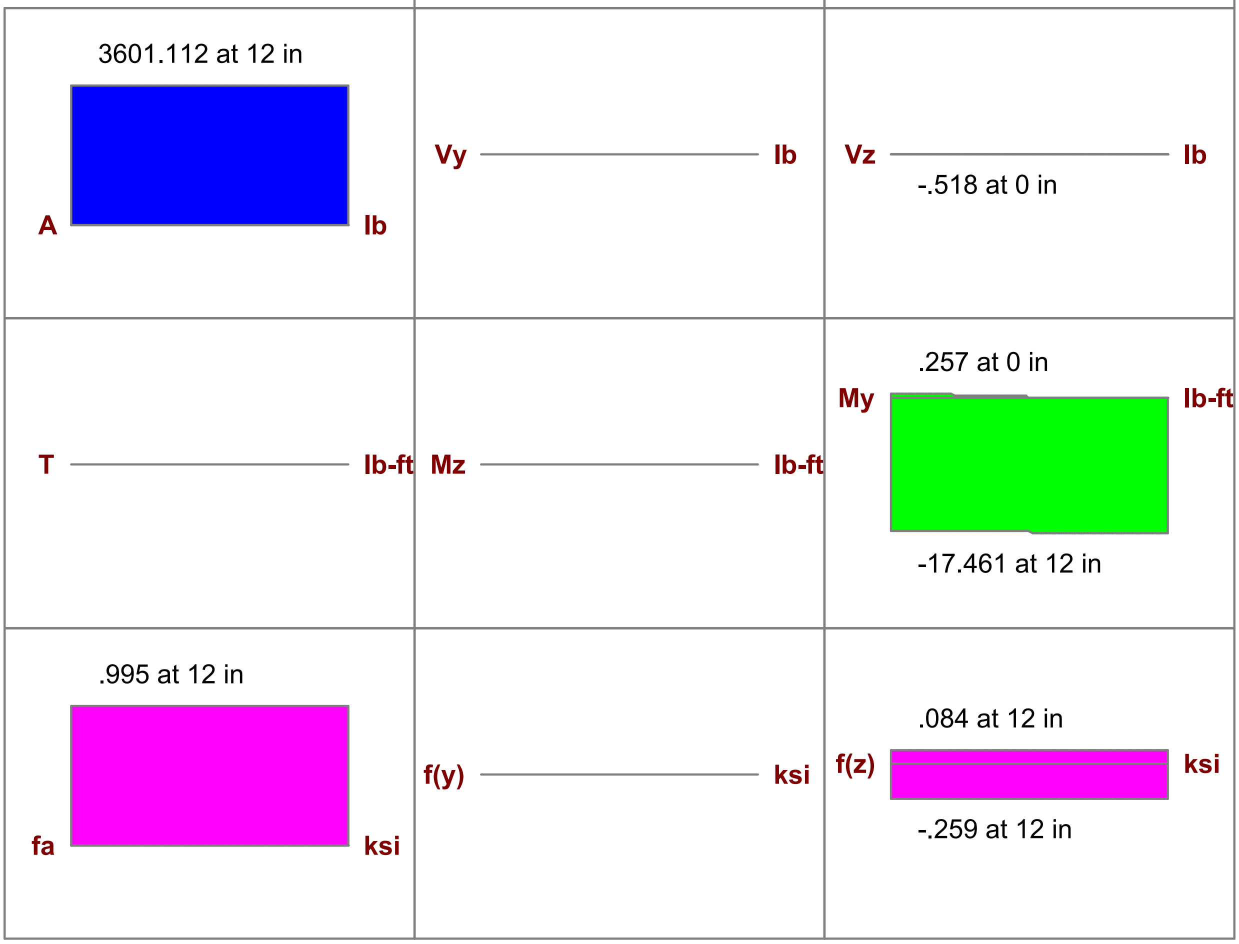
	y-y	z-z
Lb	**24 in**	**24 in**
KL/r	**38.592**	**7.843**

L Comp Top	**24 in**
L Comp Bot	**24 in**
Torque Length	**24 in**
Tau_b	**1**
Cb	**1**

Member: ***M9***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **12 in**
I Joint: **N16**
J Joint: **N14**
Envelope
Code Check: **0.128 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.128 (LC 5)**	Max Shear Check	**0.000 (z) (LC 5)**
Location	**12 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**32909.091 lb**				**E.4-1**
Mny/om	**919.601 lb-ft**				**B.5.4.2**
Mnz/om	**6420.455 lb-ft**				**B.5.4.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

	y-y	z-z
Lb	**12 in**	**12 in**
KL/r	**19.296**	**3.921**

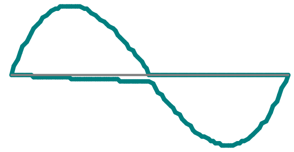
L Comp Top	**12 in**
L Comp Bot	**12 in**
Torque Length	**12 in**
Tau_b	**1**
Cb	**1**

Member: ***M10***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **12 in**
I Joint: **N14**
J Joint: **N13**
Envelope
Code Check: **0.019 (LC 5)**
Report Based On 97 Sections

Dy in

0 at 2.5 in

Dz  in

0 at 9.375 in

A lb

Vy lb

36.578 at 0 in

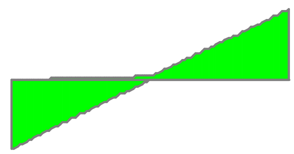
Vz  lb

-2.172 at 12 in

T lb-ft

Mz lb-ft

17.201 at 12 in

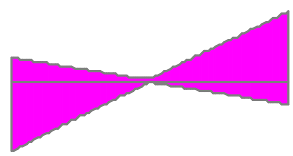
My  lb-ft

-17.461 at 0 in

fa ksi

f(y) ksi

.255 at 12 in

f(z)  ksi

-.259 at 0 in

AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.019 (LC 5)**	Max Shear Check	**0.004 (z) (LC 5)**
Location	**0 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

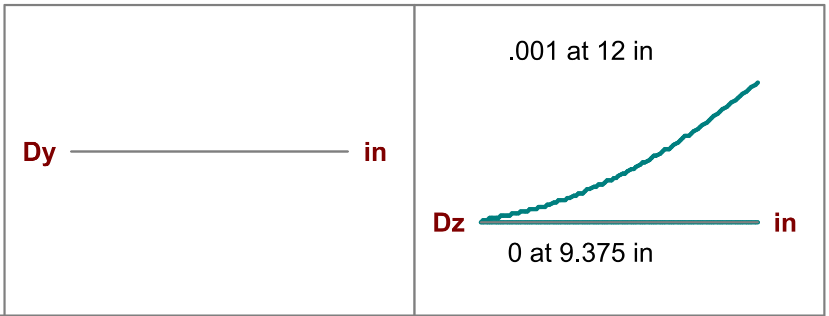
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**32909.091 lb**				**E.4-1**
Mny/om	**919.601 lb-ft**				**B.5.4.2**
Mnz/om	**6420.455 lb-ft**				**B.5.4.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

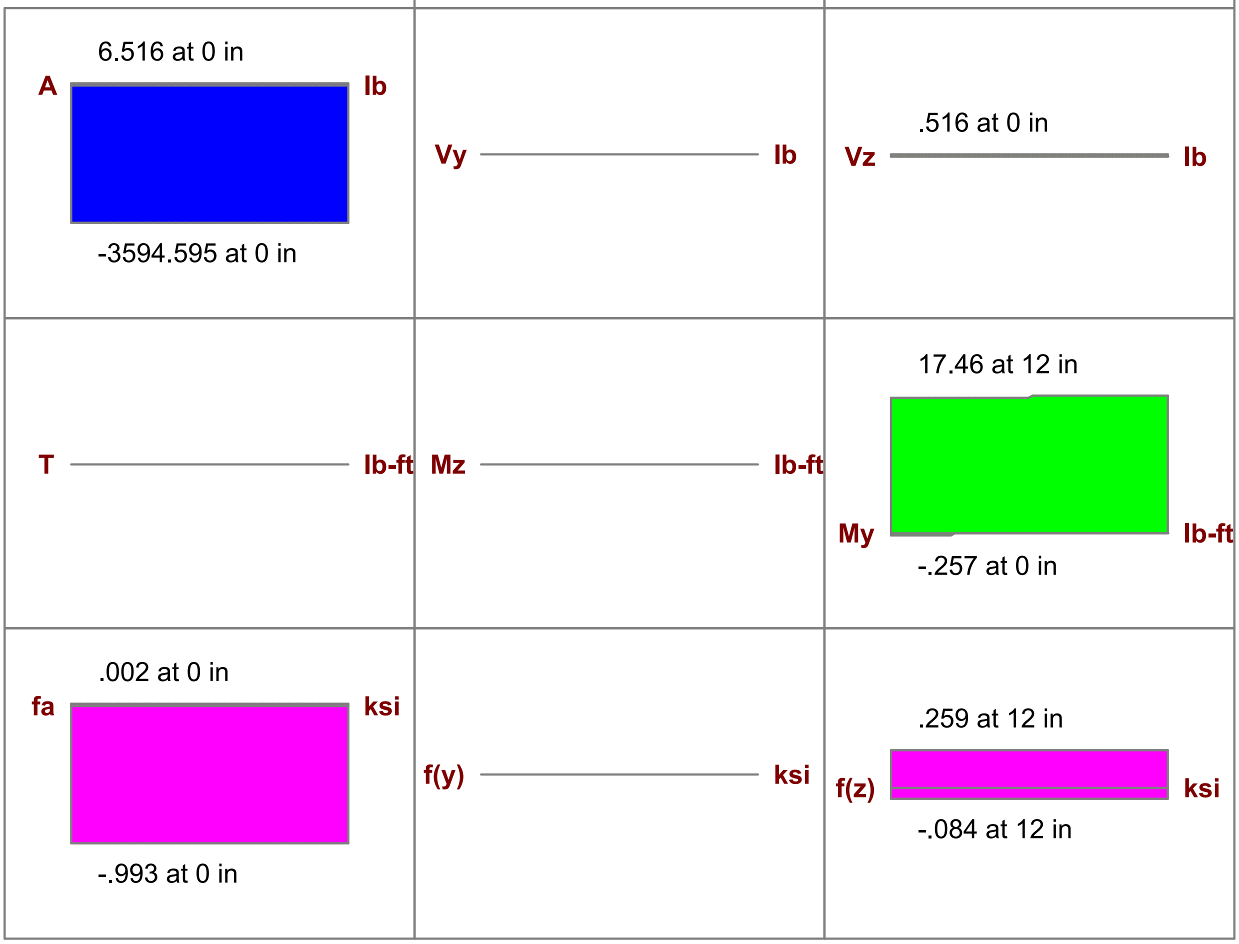
	y-y	z-z
Lb	**12 in**	**12 in**
KL/r	**19.296**	**3.921**

L Comp Top	**12 in**
L Comp Bot	**12 in**
Torque Length	**12 in**
Tau_b	**1**
Cb	**1**

Member: ***M11***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **12 in**
I Joint: **N13**
J Joint: **N15**
Envelope
Code Check: **0.128 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

Max Bending Check	**0.128 (LC 5)**	Max Shear Check	**0.000 (z) (LC 5)**
Location	**12 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

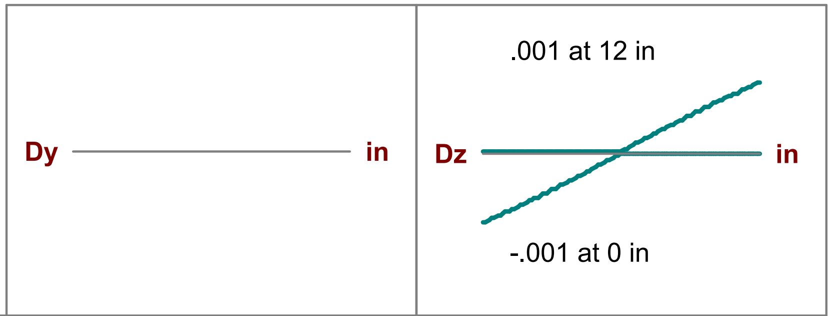
		Slender. Limit $\lambda 1$	$\lambda 2$	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**32909.091 lb**				**E.4-1**
Mny/om	**919.601 lb-ft**				**B.5.5.2**
Mnz/om	**6420.455 lb-ft**				**B.5.5.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

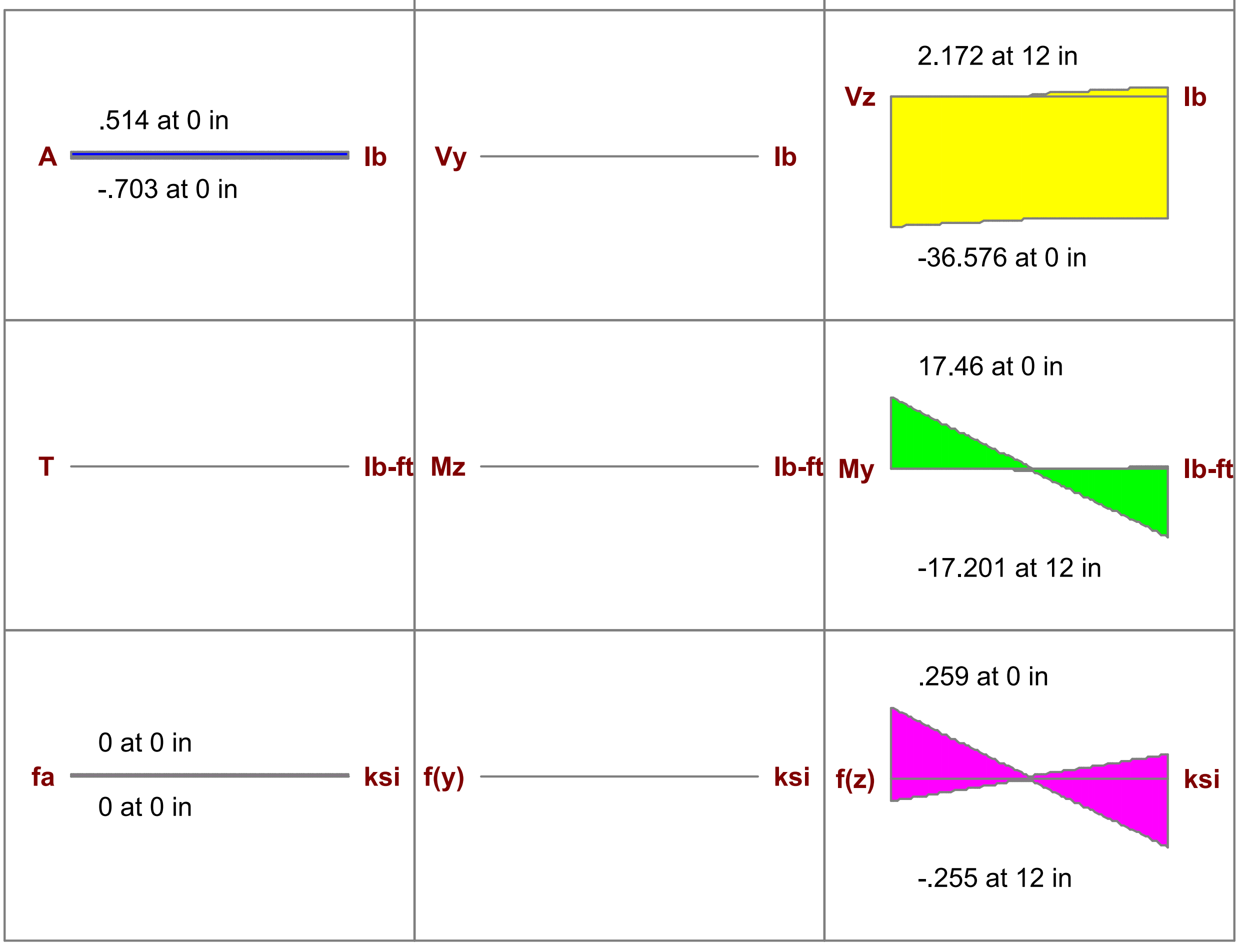
	y-y	z-z
Lb	**12 in**	**12 in**
KL/r	**19.296**	**3.921**

L Comp Top	**12 in**
L Comp Bot	**12 in**
Torque Length	**12 in**
Tau_b	**1**
Cb	**1**

Member: ***M12***

Shape: **USC8X4.25**
Material: **6061-T6 W**
Length: **12 in**
I Joint: **N15**
J Joint: **N16**
Envelope
Code Check: **0.019 (LC 5)**
Report Based On 97 Sections





AA ADM1-15: ASD - Building Code Check

-- Pu was ignored in the calculation of the unity check --

Max Bending Check	**0.019 (LC 5)**	Max Shear Check	**0.004 (z) (LC 5)**
Location	**0 in**	Location	**0 in**
Equation	**H.1-1**	Max Defl Ratio	**L/10000**

		Slender. Limit λ1	λ2	Slender. Ratio λ	Gov Eqn
Pnt/om	**32909.091 lb**				**D.2-1**
Pnc/om	**32909.091 lb**				**E.4-1**
Mny/om	**919.601 lb-ft**				**B.5.5.2**
Mnz/om	**6420.455 lb-ft**				**B.5.5.2**
Vny/om	**10909.091 lb**	**0**	**0**	**28.9**	**G.1-1**
Vnz/om	**9742.909 lb**	**19.8**	**52.6**	**5.9**	**G.3.1**

	y-y	z-z
Lb	**12 in**	**12 in**
KL/r	**19.296**	**3.921**

L Comp Top	**12 in**
L Comp Bot	**12 in**
Torque Length	**12 in**
Tau_b	**1**
Cb	**1**

Project: Rooftop Screen Rail Design Steel
Project Number: #24075
Date: 4/3/2024

Baseplate w/ Large Moment Design

(Review AISC design guide while designing)

Column Baseplate Design:

$P_{DL} := .513\ kip$ $\qquad M_{DL} := 1.36\ ft \cdot kip$
$P_{SL} := 0\ kip$ $\qquad M_{SL} := 0\ ft \cdot kip$

Weld Design to Column: $\quad d := 4\ in \qquad b := 4\ in$

$Size := 0.25\ in$

$A_{we} := Size \cdot 0.707 = 0.177\ in$

$F_{EXX} := 70\ ksi \qquad \Omega := 2.00$

$$M_a := M_{DL} + M_{SL} = 1.36\ ft \cdot kip$$

$$S_w := (b \cdot d) + \frac{d^2}{3} = 21.333\ in^2$$

$$F_{weld} := \frac{M_a}{S_w} = 0.765\ kpi$$

$$F_{nw} := 0.6 \cdot F_{EXX} = 42\ ksi$$

$$R_n := \frac{F_{nw} \cdot A_{we}}{\Omega} = 3.712\ kpi$$

$$Check_{weld} := \text{if}\left(F_{weld} < R_n, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Use 1/4" Fillet weld all around

Baseplate Size Design: $\quad f'_c := 3\ ksi \qquad \Omega_c := 2.5 \qquad d := 4\ in \qquad b := 4\ in$

$F_y := 36\ ksi \qquad t_f := .25\ in$

$d_{edge} := 1.5\ in$

1. Compute Required Strength

$M_a = 1.36\ ft \cdot kip$

$P_a := P_{DL} + P_{SL} = 0.513\ kip$

2. Choose Trial Baseplate Size

$N := 8\ in \qquad B := 10\ in$

3. Determine e & ecrit; check inequality in Eqn. 3.4.4 to determine if a solution exists

$$e := \frac{M_a}{P_a} = 31.813 \ in$$

$$f_{pmax} := \frac{(0.85 \cdot f'_c)}{\Omega_c} = 1.02 \ ksi$$

$$q_{max} := f_{pmax} \cdot B = 10.2 \ kpi$$

$$e_{crit} := \left(\frac{N}{2}\right) - \left(\frac{P_a}{2 \cdot q_{max}}\right) = 3.975 \ in$$

$$Check_{eccentricity} := \text{if}\left(e < e_{crit}, \text{“Small Moment Design”}, \text{“Large Moment Design”}\right)$$

$$Check_{eccentricity} = \text{“Large Moment Design”}$$

$$f := \left(\frac{N}{2}\right) - d_{edge} = 2.5 \ in$$

(Assuming anchor rod edge distance = 1.5")

$$A_1 := \left(f + \frac{N}{2}\right)^2 = 42.25 \ in^2$$

$$A_2 := \frac{(2 \cdot P_a \cdot (e + f))}{q_{max}} = 3.451 \ in^2$$

$$Check := \text{if}\left(A_1 > A_2, \text{“Solution Exists”}, \text{“Pick New Baseplate Size”}\right)$$

$$Check = \text{“Solution Exists”}$$

4. Determine bearing length, Y, and anchor rod tension, Ta

$$Y := \left(f + \frac{N}{2}\right) - (A_1 - A_2)^{0.5} = 0.271 \ in$$

$$T_a := (q_{max} \cdot Y) - P_a = 2.253 \ kip$$

5. Determine minimum plate thickness

At bearing interface:

$$m := \frac{(N - (0.95 \cdot d))}{2} = 2.1 \ in$$

$$E_1 := Y \cdot \left(m - \frac{Y}{2}\right) \qquad E_2 := \frac{f_{pmax}}{F_y}$$

$$t_{p1} := 2.58 \cdot \left(E_1 \cdot E_2\right)^{0.5} = 0.317 \; in$$

$$t_{p2} := 1.83 \cdot m \cdot {E_2}^{0.5} = 0.647 \; in$$

$$t_{preq1} := \text{if}\left(Y < m, t_{p1}, t_{p2}\right) = 0.317 \; in$$

$$t_{preq1} = 0.317 \; in$$

At tension interface:

$$x := \frac{N}{2} - \frac{d}{2} + \frac{t_f}{2} - d_{edge} = 0.625 \; in$$

$$t_{preq2} := 2.58 \cdot \left(\frac{T_a \cdot x}{B \cdot F_y}\right)^{0.5} = 0.161 \; in$$

$$t_{preq2} = 0.161 \; in$$

Check the thickness using the value of n:

$$n := \frac{B - (0.8 \cdot b)}{2} = 3.4 \; in$$

$$E_1 := Y \cdot \left(n - \frac{Y}{2}\right) \qquad E_2 := \frac{f_{pmax}}{F_y}$$

$$t_{p1} := 2.58 \cdot \left(E_1 \cdot E_2\right)^{0.5} = 0.409 \; in$$

$$t_{p2} := 1.83 \cdot n \cdot {E_2}^{0.5} = 1.047 \; in$$

$$t_{preq3} := \text{if}\left(Y < n, t_{p1}, t_{p2}\right) = 0.409 \; in$$

$$t_{preq3} = 0.409 \; in$$

$$t_{required} := \max\left(t_{preq1}, t_{preq2}, t_{preq3}\right) = 0.409\ in$$

$$t_{required} = 0.409\ in$$

Use 8"x10"x.75" Baseplate w/ (4) 3/4" anchor bolts w/ a 1.5" edge distance to center of hole at corners of base plate

Project: **Rooftop Screen Rail Design Aluminum**
Project Number: #24075
Date: **4/3/2024**

Baseplate w/ Large Moment Design

(Review AISC design guide while designing)

Column Baseplate Design:

$P_{DL} := .513\ kip$ $\qquad M_{DL} := 1.36\ ft \cdot kip$
$P_{SL} := 0\ kip$ $\qquad M_{SL} := 0\ ft \cdot kip$

Weld Design to Column: $d := 4\ in$ $\qquad b := 4\ in$

$Size := 0.25\ in$

$A_{we} := Size \cdot 0.707 = 0.177\ in$

$F_{EXX} := 70\ ksi$ $\qquad \Omega := 2.00$

$$M_a := M_{DL} + M_{SL} = 1.36\ ft \cdot kip$$

$$S_w := (b \cdot d) + \frac{d^2}{3} = 21.333\ in^2$$

$$F_{weld} := \frac{M_a}{S_w} = 0.765\ kpi$$

$$F_{nw} := 0.6 \cdot F_{EXX} = 42\ ksi$$

$$R_n := \frac{F_{nw} \cdot A_{we}}{\Omega} = 3.712\ kpi$$

$$Check_{weld} := \text{if}\left(F_{weld} < R_n, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Use 1/4" Fillet weld all around

Baseplate Size Design: $f'_c := 3\ ksi$ $\qquad \Omega_c := 2.5$ $\qquad d := 4\ in$ $\qquad b := 4\ in$

$F_y := 35\ ksi$ $\qquad t_f := .25\ in$

$d_{edge} := 1.5\ in$

1. Compute Required Strength

$M_a = 1.36\ ft \cdot kip$

$P_a := P_{DL} + P_{SL} = 0.513\ kip$

2. Choose Trial Baseplate Size

$N := 8\ in$ $\qquad B := 10\ in$

3. Determine e & ecrit; check inequality in Eqn. 3.4.4 to determine if a solution exists

$$e := \frac{M_a}{P_a} = 31.813\ in$$

$$f_{pmax} := \frac{(0.85 \cdot f'_c)}{\Omega_c} = 1.02\ ksi$$

$$q_{max} := f_{pmax} \cdot B = 10.2\ kpi$$

$$e_{crit} := \left(\frac{N}{2}\right) - \left(\frac{P_a}{2 \cdot q_{max}}\right) = 3.975\ in$$

$$Check_{eccentricity} := \text{if}\left(e < e_{crit}, \text{“Small Moment Design”}, \text{“Large Moment Design”}\right)$$

$$Check_{eccentricity} = \text{“Large Moment Design”}$$

$$f := \left(\frac{N}{2}\right) - d_{edge} = 2.5\ in$$ (Assuming anchor rod edge distance = 1.5")

$$A_1 := \left(f + \frac{N}{2}\right)^2 = 42.25\ in^2$$

$$A_2 := \frac{(2 \cdot P_a \cdot (e + f))}{q_{max}} = 3.451\ in^2$$

$$Check := \text{if}\left(A_1 > A_2, \text{“Solution Exists”}, \text{“Pick New Baseplate Size”}\right)$$

$$Check = \text{“Solution Exists”}$$

4. Determine bearing length, Y, and anchor rod tension, Ta

$$Y := \left(f + \frac{N}{2}\right) - (A_1 - A_2)^{0.5} = 0.271\ in$$

$$T_a := (q_{max} \cdot Y) - P_a = 2.253\ kip$$

5. Determine minimum plate thickness

At bearing interface:

$$m := \frac{(N - (0.95 \cdot d))}{2} = 2.1\ in$$

$$E_1 := Y \cdot \left(m - \frac{Y}{2}\right) \qquad E_2 := \frac{f_{pmax}}{F_y}$$

$$t_{p1} := 2.58 \cdot \left(E_1 \cdot E_2\right)^{0.5} = 0.321 \; \boldsymbol{in}$$

$$t_{p2} := 1.83 \cdot m \cdot E_2^{\;0.5} = 0.656 \; \boldsymbol{in}$$

$$t_{preq1} := \text{if}\left(Y < m, t_{p1}, t_{p2}\right) = 0.321 \; \boldsymbol{in}$$

$$t_{preq1} = 0.321 \; \boldsymbol{in}$$

At tension interface:

$$x := \frac{N}{2} - \frac{d}{2} + \frac{t_f}{2} - d_{edge} = 0.625 \; \boldsymbol{in}$$

$$t_{preq2} := 2.58 \cdot \left(\frac{T_a \cdot x}{B \cdot F_y}\right)^{0.5} = 0.164 \; \boldsymbol{in}$$

$$t_{preq2} = 0.164 \; \boldsymbol{in}$$

Check the thickness using the value of n:

$$n := \frac{B - (0.8 \cdot b)}{2} = 3.4 \; \boldsymbol{in}$$

$$E_1 := Y \cdot \left(n - \frac{Y}{2}\right) \qquad E_2 := \frac{f_{pmax}}{F_y}$$

$$t_{p1} := 2.58 \cdot \left(E_1 \cdot E_2\right)^{0.5} = 0.414 \; \boldsymbol{in}$$

$$t_{p2} := 1.83 \cdot n \cdot E_2^{\;0.5} = 1.062 \; \boldsymbol{in}$$

$$t_{preq3} := \text{if}\left(Y < n, t_{p1}, t_{p2}\right) = 0.414 \; \boldsymbol{in}$$

$$t_{preq3} = 0.414 \; \boldsymbol{in}$$

$$t_{required} := \max\left(t_{preq1}, t_{preq2}, t_{preq3}\right) = 0.414\ in$$

$$t_{required} = 0.414\ in$$

Use 8"x10"x.75" Baseplate w/ (4) 3/4" anchor bolts w/ a 1.5" edge distance to center of hole at corners of base plate