



FLORIDA PRODUCT APPROVAL – SLIDER DOOR ASSEMBLY

2023 FLORIDA BUILDING CODE, 8th Edition
ASCE 7-22 MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES
DESIGN SOFTWARE CREATED BY METAL BUILDING SOFTWARE
SLIDER DOOR ASSEMBLY TO BE USED ON RISK CATEGORY II BUILDINGS
SLIDER DOOR ASSEMBLY TO BE USED IN WALL ZONE 4

(Structural brace will be provided and shall be installed at 7' above floor level in the occurrence of a high wind advisory)

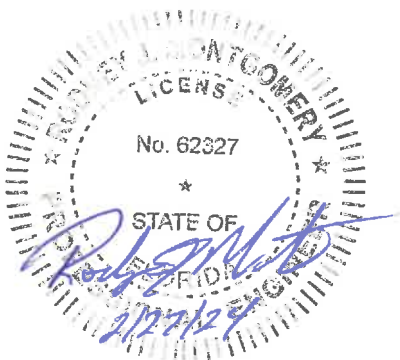
DOOR DIMENSIONS: DOOR LEAF WIDTH UP TO: **21'0"**

DOOR LEAF HEIGHT UP TO: **40'0"**

WIND LOAD DATA:	Basic wind velocity	= 180 MPH	(Fig. 26.5-1B)
	Wind directionality factor (Kd)	= 0.85	(Table 26.6-1)
	Exposure category	= D	(Section 26.7)
	Velocity Pressure Exp. Coeff. (Kz)	= 1.22	(Table 26.10-1)
	Topographic factor (Kzt)	= 1.00	(Section 26.8.2)
	Ground Elevation Factor (Ke)	= 1.00	(Table 26.9-1)
	Gust effect factor	= 0.85	(Section 26.11.1)
	Enclosure Classification	= Enclosed	(Section 26.12)
	Internal pressure coefficient (GCpi)	= 0.18	(Table 26.13-1)

WIND LOAD CALCULATIONS AND COEFFICIENTS:

$$\begin{aligned}\text{Velocity Pressure} = q_z &= .00256 K_z K_{zt} K_d K_e V^2 && (26.10-1) \\ &= .00256 (1.22)(1.00)(0.85)(1.00)(180)^2 \\ &= 86.0 \text{ psf}\end{aligned}$$





DESIGN OF SIDE RAILS: Maximum effective wind area = 174 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.76$ $G_{Cpi} = -0.18$ (Figure 30.3-1)
 Zone 4 Leeward : $G_{Cp} = 0.86$ $G_{Cpi} = -0.18$ (controls design)

Wind pressure = $p = q_z (G_{Cp} - G_{Cpi})$
 $= 86.0 (.86 - (-0.18))$
 $= 89.5 \text{ psf}$

Wind Pressure (ASD) $= 89.5 (0.6)$
 $= 53.7 \text{ psf}$

Unit load calc = $(53.7 \text{ lbs/sq.ft.})(10.5 \text{ ft.})$
 $= 563.85 \text{ lb/ft} = .0470 \text{ k/in}$

Deflection calculation = $5wl^4 / 384 EI$
 $= 5 (0.0470)(396)^4 / (384)(29000)(162)$
 $= 3.20 \text{ in} < 3.3 \text{ in} \quad (L/120) \text{ OK!!!}$

Design Check for Vertical Side Rails:

STRUCTURAL CODE

Design Basis - Working Stress (WS)
 Hot Rolled Steel - AISC16

MEMBER

Member Part - C12X30
 Web Depth (in) - 11.000
 Web Thick (in) - 0.510
 Flange Width (in) - 3.170
 Top Flange Thick(in)- 0.501
 Bot Flange Thick(in)- 0.501
 Yield Flange (ksi) - 50.000
 Yield Web (ksi) - 50.000
 Stiffener Space(in) - 0.000

UNBRACED LENGTH

Major Axis (ft) - 33.000
 Minor Axis Axial(ft)- 4.000
 Minor Axis Top (ft) - 4.000
 Minor Axis Bot (ft) - 4.000
 Cb Top - 1.000
 Cb Bottom - 1.000

CONSTANTS

Tension Coefficient - 1.000

ACTION

Axial (k) - 3.000
 Shear (k) - 11.250
 Moment (f-k) - 76.800

		UNITS(k , f-k)			
		CALC	LIMIT	RATIO	EQUATION
ACTION					
-----	-----	-----	-----	-----	-----
Axial		3.00	141.63	0.02	E3-1
Shear		11.25	109.96	0.10	G2-1
Bending_Top	(Tension)	76.80	84.06	0.91	F2-1
Bending_Bot	(Compression)	76.80	78.36	0.98	F2-2
Axial+Bending_Top				0.90	
Axial+Bending_Bot				0.99	H1-1b

MEMBER CLASSIFICATION

Axial : Web = Non-Slender, Flange = Non-Slender
 Bending: Web = Compact , Flange = Compact

SLENDERNESS

Major Axis 92.31
 Minor Axis 62.91



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CALCULATIONS

DESIGN BASIS = WORKING STRESS (ASD)

SECTION SIZE

Member Part	=	C12X30
d, Total Depth	=	12.002 in
h _w , Web Depth	=	11.000 in
t _w , Web Thickness	=	0.510 in
b _f , Flange Width	=	3.170 in
t _{ft} , Top Flange Thickness	=	0.501 in
t _{fb} , Bottom Flange Thickness	=	0.501 in
Stiffener Use	=	N
a, Stiffener Spacing	=	0.000 in

MATERIAL

F _{yw} , Web Yield Stress	=	50.00 ksi
F _{uw} , Web Ultimate Stress	=	65.00 ksi
F _{yf} , Flange Yield Stress	=	50.00 ksi
F _{uf} , Flange Ultimate Stress	=	65.00 ksi

CONSTANTS

E, Modulus of Elasticity	=	29000.00 ksi
G, Shear Modulus	=	11200.00 ksi
Ω _v , Safety Factor for Shear	=	1.67
Ω _c , Safety Factor for Axial Compression	=	1.67
Ω _b , Safety Factor for Bending	=	1.67
C _a = A _e /A _g , Tension Net Area Coefficient	=	1.00

UNBRACED LENGTHS

L _x , Major Axis	=	33.0000 ft
L _y , Minor Axis (Axial)	=	4.0000 ft
L _z , Minor Axis (Axial)	=	4.0000 ft
L _{bt} , Minor Axis, Top Flange	=	4.0000 ft
L _{fb} , Minor Axis, Bottom Flange	=	4.0000 ft
C _b , Bending Coefficient	=	1.00

LOADING - WORKING LOADS

P _x , Axial (Compression)	=	3.00 k
V _x , Shear	=	11.25 k
M _x , Moment (major axis, bottom flange in compression)	=	76.80 f-k

MEMBER CLASSIFICATION

AXIAL

e = √(E/F _{yw})	=	24.083
λ _{pw} = Not applicable		
λ _{rw} = 1.49*e	=	35.88
λ _w = h _w /t _w = 21.6 < 35.9		Web is Non-Slender
λ _{pf} = Not applicable		
K _c = 4/√(h _w /t _w) ≥ 0.35, ≤ 0.76	=	0.7600
λ _{rf} = 0.64*√(K _c *E/F _{yf})	=	13.44
λ _{fo} = b _{fo} /t _{fo} = 6.33 < 13.44		Top Flange is Non-Slender
λ _{fi} = b _{fi} /t _{fi} = 6.33 < 13.44		Bottom Flange is Non-Slender

BENDING

e = √(E/F _{yw})	=	24.083
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λ _{pw} = 3.76*e,	Compact limit	=	90.553
λ _{rw} = 5.70*e,	Non-compact limit	=	137.274
λ _w = h _w /t _w = 21.6 < 90.6		Web is Compact	
λ _{pf} = 0.38*√(E/F _{yf}),	Compact limit	=	9.152



$$\begin{aligned}
 K_c &= 4/\sqrt{(h_w/t_w)} \geq 0.35, \leq 0.76 &= & 0.760 \\
 S_{xt}/S_{xc} & &= & 1.000 \\
 \text{Since } S_{xt}/S_{xc} &= 1.000 \geq 0.7, & & \\
 F_L &= 0.7 \cdot F_{yf} &= & 35.000 \text{ ksi} \\
 \lambda_{rf} &= 0.95 \cdot \sqrt{(K_c \cdot E/F_L)}, \text{ Non-compact limit} &= & 23.839 \\
 \lambda_{fo} &= b_{fo}/t_{fo} = 6.33 < 9.15 & \text{Top Flange is} & \text{Compact} \\
 \lambda_{fi} &= b_{fi}/t_{fi} = 6.33 < 9.15 & \text{Bottom Flange is} & \text{Compact}
 \end{aligned}$$

SECTION PROPERTIES

$$\begin{aligned}
 d, \text{ overall member depth} &= 12.002 \text{ in} \\
 h_o, \text{ distance between flange centroids} &= 11.501 \text{ in} \\
 A_g, \text{ gross area} &= 8.820 \text{ in}^2 \\
 Y_{bar}, \text{ centroid, from bottom} &= 6.001 \text{ in} \\
 I_x, \text{ major axis moment of inertia} &= 162.0000 \text{ in}^4 \\
 S_{xt}, \text{ elastic section modulus, tension flange} &= 27.000 \text{ in}^3 \\
 S_{xc}, \text{ elastic section modulus, compression flange} &= 27.000 \text{ in}^3 \\
 r_x, \text{ major axis radius of gyration} &= 4.290 \text{ in} \\
 I_y, \text{ minor axis moment of inertia} &= 5.1400 \text{ in}^4 \\
 S_y, \text{ elastic section modulus for y-axis} &= 2.060 \text{ in}^3 \\
 r_y, \text{ minor axis radius of gyration} &= 0.763 \text{ in} \\
 Z_x, \text{ plastic section modulus} &= 33.693 \text{ in}^3 \\
 J, \text{ Torsional Constant} &= 0.752917 \text{ in}^4 \\
 r_t, \text{ torsion radius of gyration} &= 0.732 \text{ in} \\
 I_{yc}, \text{ minor axis moment of inertia in compression} &= 1.391 \text{ in}^4 \\
 I_{yt}, \text{ minor axis moment of inertia in tension} &= 1.391 \text{ in}^4 \\
 C_w, \text{ warping constant} &= 150.666 \text{ in}^6 \\
 x_o, \text{ shear center with respect to centroid of section} &= -0.873 \text{ in} \\
 y_o, \text{ shear center with respect to centroid of section} &= 0.000 \text{ in} \\
 r_o, \text{ polar radius of gyration about shear center} &= 4.326 \text{ in} \\
 \beta_x, \text{ asymmetry parameter} &= -4.745 \text{ in}
 \end{aligned}$$

SHEAR STRENGTH

UNSTIFFENED WEB, (Chapter G2.1)

$$\begin{aligned}
 k_v &= 5.3400 \\
 h_w/t_w &= 21.57 \\
 1.10 \cdot \sqrt{(k_v \cdot E/F_{yw})} &= 61.22 \\
 \text{Since } h_w/t_w &\leq 1.10 \cdot \sqrt{(k_v \cdot E/F_{yw})}, & & \\
 C_{v1} &= 1.000 & \text{(G2-3)} \\
 V_n &= 0.60 \cdot F_{yw} \cdot d \cdot t_w \cdot C_{v1} &= & 183.631 \text{ k} & \text{(G2-1)} \\
 \text{Limit} &= V_n/\Omega_v &= & 109.958 \text{ k} \\
 \text{Ratio} &= V_r \cdot \Omega_v/V_n &= & 0.1023
 \end{aligned}$$

AXIAL STRENGTH

COMPRESSION OF NON-SLENDER ELEMENTS, (Chapter E, E3)

$$\begin{aligned}
 L_{cx}, \text{ major axis length} &= K \cdot L_x &= & 33.000 \text{ ft} \\
 L_{cy}, \text{ minor axis length} &= K \cdot L_y &= & 4.000 \text{ ft} \\
 L_{cz}, \text{ longitudinal axis length} &= K \cdot L_z &= & 4.000 \text{ ft} \\
 L_{cx}/r_x &= 92.308 \\
 L_{cy}/r_y &= 62.910 \\
 F_e &= \pi^2 \cdot E / (L_c/r)^2 &= & 33.59 \text{ ksi} \\
 4.71 \cdot \sqrt{(E/F_{yf})} &= 113.43 \\
 \text{Since maximum } L_c/r &< 4.71 \cdot \sqrt{(E/F_{yf})} & & \\
 F_{cr} &= (0.658^{F_{yf}/F_e}) \cdot F_{yf} &= & 26.82 \text{ ksi} & \text{(E3-2)} \\
 P_n &= A_g \cdot F_{cr} &= & 236.520 \text{ k} & \text{(E7-1)}
 \end{aligned}$$

/*

LOAD RATIO FOR COMPRESSION

$$\text{Ratio} = P_r / (P_n / \Omega_c) = 0.0212$$

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BENDING STRENGTH



COMPRESSION FLANGE (Chapter F, F2)		
Flange in compression	=	Bottom
h_c	=	11.000 in
h_p	=	11.000 in
FLANGE YIELDING		
$M_n = M_p = F_{yf} * Z_x$	=	140.388 f-k (F2-1)
LATERAL-TORSIONAL BUCKLING		
$r_{ts} = \sqrt{(I_y * C_w) / S_x}$	=	1.015 in (F2-7)
$L_p = 1.76 * r_{ts} * \sqrt{(E / F_{yf})}$	=	32.341 in (F2-5)
Member type is U, therefore		
$C = h_o / 2 * \sqrt{(I_y / C_w)}$	=	1.062
$C2 = J * C / (S_{xc} * h_o)$	=	0.002575
$C1 = \sqrt{C2 * [1 + \sqrt{1 + 6.76 * (0.7 * F_{yf} / E / C2)^2}]}$	=	0.0815
$L_r = 1.95 * r_{ts} * E / (0.7 * F_{yf}) * C1$	=	133.610 in (F2-6)
Since $L_p < L_b \leq L_r$, Use this formula for M_n		
$M_p = Z_x * F_{yf}$	=	140.388 f-k
$M_n = C_b * [M_p - (M_p - 0.7 * F_{yf} * S_x) * (L_b - L_p) / (L_r - L_p)] \leq M_p$	=	130.857 f-k (F2-2)
NOMINAL STRENGTH		
M_n is the minimum of Flange Yielding and Lateral-Torsional Buckling		
$M_n = 130.857 \text{ f-k}$		
Limit = M_n / Ω_b	=	78.357 f-k
Ratio = $M_x * \Omega_b / M_n$	=	0.980
TENSION FLANGE YIELDING, (Chapter F, F2)		
Flange in tension	=	Top
Since $S_{xt} > S_{xc}$, the limit state of tension flange yielding does not apply		
$M_n = F_{yf} * Z_x$	=	140.388 f-k (F2-1)
Limit = M_n / Ω_b	=	84.064 f-k
Ratio = $M_x * \Omega_b / M_n$	=	0.914
AXIAL + BENDING STRENGTH (Chapter H)		
P_r , required axial strength	=	3.000 k
P_c , minimum compressive strength	=	141.629 k
M_r , required bending strength	=	76.800 f-k
$M_c = M_n / \Omega_b$, allowable bending strength, compression flange	=	78.357 f-k
$M_{ct} = M_n / \Omega_b$, allowable bending strength, tension flange	=	84.064 f-k
A_e , gross area	=	8.820 in ²
COMPRESSION FLANGE		
H1-1 FORMULA		
For $P_r / P_c < 0.20$,		
Ratio = $P_r / (2 * P_c) + M_r / M_c$	=	0.9907 (H1-1b)
H1-2 FORMULAS		
Since this is a rolled compact double symmetric section and single axis flexure, we may use Section H1.3 in lieu of the H1-1 formulas.		
OUT-OF-PLANE BUCKLING		
Ratio = $(P_r / P_{co}) * (1.5 - 0.5 * P_r / P_{co}) + (M_r / M_{cx})^2$	=	0.9922 (H1-2)
IN-PLANE INSTABILITY		
For $P_r / P_{ci} < 0.20$,		
Ratio = $P_r / (2 * P_{ci}) + M_r / M_c$	=	0.9907 (H1-1b)
MAXIMUM RATIO OF H1-2 FORMULAS		
Ratio	=	0.9922
MINIMUM RATIO OF H1-1 and H1-2 FORMULAS		
H1-1 formula controls		
Ratio	=	0.9907
TENSION FLANGE		
$f_a = P_r / A_e$, Axial Stress	=	0.34 ksi
/*		
$f_b = M_r / S_{xt}$, Bending tension stress	=	34.13 ksi
$f_a \leq f_b$, therefore:		
Ratio = $(M_r / M_{ct}) - (P_r / A_e) / (M_{ct} / S_{xt})$	=	0.9045



DESIGN OF INTERMEDIATE RAILS (3'6 c/c): Maximum effective wind area = 72 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.86$ $G_{Cpi} = -0.18$ (Figure 30.3-1)
 Zone 4 Leeward : $G_{Cp} = 0.94$ $G_{Cpi} = -0.18$ (leeward controls design)

$$\begin{aligned} \text{Wind pressure} &= p = q_z (G_{Cp} - G_{Cpi}) \\ &= 86.0 (.94 - (-0.18)) \\ &= 96.3 \text{ lbs/sq. ft.} \end{aligned}$$

$$\begin{aligned} \text{Wind Pressure (ASD)} &= 96.3 (0.6) \\ &= 57.8 \text{ psf} \end{aligned}$$

$$\begin{aligned} \text{Unit load calc} &= (57.8 \text{ lbs/sq.ft.})(3.5 \text{ ft.}) \\ &= 202.3 \text{ lb/ft} = .0169 \text{ k/in} \end{aligned}$$

$$\begin{aligned} \text{Deflection calculation} &= 5wl^4 / 384 EI \\ &= 5 (0.0169)(246)^4 / (384)(29000)(25.6) \\ &= 1.08 \text{ in} < 2.05 \text{ in (L/120)} \quad \text{OK!!!} \end{aligned}$$

Design Check for Intermediate Rail:

DESIGN SUMMARY FOR COLD FORMED MEMBERS

INPUT		OUTPUT				
CODE	= WS NAUS 16					
MEMBER						
Member Part	= 10X3C12					
Web Depth, ho	= 10.000 in	UNITS (k , f-k)				
Thickness, t	= 0.1020 in	ACTION	CALC	LIMIT	RATIO	EQUATION
Top Flg Width, b	= 3.000 in	-----	-----	-----	-----	-----
Bot Flg Width	= 3.000 in	Axial	0.00	22.73	0.00	E3.1-1
Lip Length, D	= 0.750 in					
Inside Radius, R	= 0.188 in	Shear	2.07	10.03	0.21	G2.1-3a
Angle	= 90.000 deg					
Steel Yield	= 55.000 ksi	Bending-Top	10.63	13.09	0.81	F3.1-1
UNBRACED LENGTH		(Tension)				
Major Axis	= 20.500 ft					
Minor Axis Axl	= 6.667 ft					
Minor Axis top	= 6.667 ft	Bending-Bot	10.63	11.84	0.90	F4.1-2
Minor Axis bot	= 6.667 ft	(Compression)				
Cb	= 1.000					
M1/M2	= 0.000	Axial+Bending_Top			0.81	H1.2-1
Strength Increase	= 1.000					
ACTION (Working Loads)		Axial+Bending_Bot			0.90	H1.2-1
Axial	= 0.000 k					
Shear	= 2.074 k	Shear+Bending			0.84	H2-1
Moment	= 10.630 f-k					
WEB CRIPPLING						
Reaction	= 2.000 k	Web Crippling	2.00	2.73	0.73	G5-1
Bearing Width	= 4.000 in					
Bearing Location	= End	WC+Bending	0.87	0.78	1.11	H3-1a
Z section	= Single					



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SECTION PROPERTIES

GROSS SECTION PROPERTIES

Weight	=	5.900	lb/ft
Ag, Area	=	1.73400	in2
Ybar, centroid, from bottom	=	5.000	in
Ixx	=	25.578	in4
Sxx Top	=	5.116	in3
Sxx Bot	=	5.116	in3
Rxx, major axis radius of gyration	=	3.820	in
Iyc Top	=	1.0250	in4
Iyc Bot	=	1.0250	in4
Syy	=	0.9430	in3
Ryy, minor axis radius of gyration	=	1.081	in
J, Torsional Constant	=	0.006080	in4
Cw, Warping Constant	=	42.86	in6
Xo	=	-2.130	in
Ro	=	4.500	in
Xbar	=	0.762	in

SECTION PROPERTIES ON FLANGE AND LIP

Af	=	0.36689	in2
xo	=	1.167	in
hx	=	-1.731	in
hy	=	-0.068	in
Jf	=	0.00127	in4
Cwf	=	0.00	in6
Ixf	=	0.01018	in4
Iyf	=	0.32748	in4
Ixyf	=	0.02909	in4

CONSTANTS

Nu	=	0.30	
E	=	29500.0	ksi
G	=	11300.0	ksi

SHEAR STRENGTH (Section G2.1, G2.3)

k_v , no web reinforcement	=	5.34	
h , web flat, $= d - 2(R + t)$	=	9.4210	in
A_w , web area, $= h * t$	=	0.9609	in ²
$F_{cr} = \pi^2 * E * k_v / (12 * (1 - \mu^2) * (h/t)^2)$	=	16.693	ksi (G2.3-2)
$V_{cr} = A_w * F_{cr}$	=	16.041	k (G2.3-1)
$V_y = 0.6 * A_w * F_y$	=	31.711	k (G2.1-5)
$\lambda_v = \sqrt{V_y / V_{cr}}$	=	1.4060	(G2.1-4)
For $\lambda_v > 1.227$			
$V_n = V_{cr}$	=	16.041	k (G2.1-3a)
Ω_v	=	1.60	
Limit $= V_n / \Omega_v$	=	10.026	k (B3.2.1-2)

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AXIAL STRENGTH

COMPRESSION: (Section E1, E2, E3, E4, I6)

MAJOR AXIS:

YIELDING AND GLOBAL BUCKLING (Section E2)

L	=	246.000	in
KL/r _x	=	64.40	
$F_{cre1} = \pi^2 * E / (K * L_x / r_x)^2$	=	70.21	ksi (E2.1-1)
$\sigma_{ex} = \pi^2 * E / (K * L_x / r_x)^2$	=	70.21	ksi (E2.2-6)
$\sigma_t = 1 / (A * r_o^2) * (G * J + \pi^2 * E * C_w / (K * L_x)^2)$	=	57.49	ksi (E2.2-5)
$\beta = 1 - (x_o / r_o)^2$	=	0.7760	(E2.2-3)
Term1 $= (\sigma_{ex} + \sigma_t) - (\sqrt{((\sigma_{ex} + \sigma_t)^2 - 4 * \beta * \sigma_{ex} * \sigma_t)})$	=	66.22	ksi
$F_{cre2} = 1 / (2\beta) * [Term1]$	=	42.67	ksi (E2.2-1)



$$\begin{aligned}
 F_{cre} &= \text{MIN} (F_{cre1} \text{ and } F_{cre2}) &= & 42.67 \text{ ksi} & (\text{E2.2-1}) \\
 \lambda_c &= \sqrt{(F_y / F_{cre})} &= & 1.1353 & (\text{E2-4}) \\
 \text{Since } \lambda_c &\leq 1.5, \\
 F_n &= 0.658 \lambda_c^2 F_y &= & 32.07 \text{ ksi} & (\text{E2-2}) \\
 P_n &= A_g * F_n &= & 55.61 \text{ k} & (\text{E2-1}) \\
 \Omega_c & &= & 1.80 \\
 \text{Limit} &= P_n / \Omega_c &= & 30.89 \text{ k} & (\text{B3.2.1-2})
 \end{aligned}$$

LOCAL BUCKLING (Section E3)

$$\begin{aligned}
 &\text{From Section E2,} \\
 F_n & &= & 32.07 \text{ ksi} & (\text{E2-2}) \\
 &\text{From Axial Effective Section Properties report} \\
 A_e &= A_g - t * (\text{sum of lost width}) &= & 1.2760 \text{ in}^2 \\
 P_{n1} &= A_e * F_n &= & 40.92 \text{ k} & (\text{E3.1-1}) \\
 \Omega_c & &= & 1.80 \\
 \text{Limit} &= P_{n1} / \Omega_c &= & 22.73 \text{ k} & (\text{B3.2.1-2})
 \end{aligned}$$

MINOR AXIS:

LOCAL BUCKLING (Section E3)

$$\begin{aligned}
 L & &= & 80.000 \text{ in} \\
 KL / r_y & &= & 74.01 \\
 F_{cre} &= \pi^2 E / (K * L_x / r_x)^2 &= & 53.16 \text{ ksi} & (\text{E2.1-1}) \\
 \lambda_c &= \sqrt{(F_y / F_{cre})} &= & 1.0172 & (\text{E2-4}) \\
 \text{Since } \lambda_c &\leq 1.5, \\
 F_n &= 0.658 \lambda_c^2 F_y &= & 35.67 \text{ ksi} & (\text{E2-2}) \\
 &\text{From Axial Effective Section Properties report} \\
 A_e &= A_g - t * (\text{sum of lost width}) &= & 1.2307 \text{ in}^2 \\
 P_{n1} &= A_e * F_n &= & 43.90 \text{ k} & (\text{E3.1-1}) \\
 \Omega_c & &= & 1.80 \\
 \text{Limit} &= P_{n1} / \Omega_c &= & 24.39 \text{ k} & (\text{B3.2.1-2})
 \end{aligned}$$

SUMMARY:

MAJOR AXIS:

$$\begin{aligned}
 \text{Yield and other buckling} & &= & 22.73 \text{ k} & (\text{E3.1-1})
 \end{aligned}$$

MINOR AXIS:

$$\begin{aligned}
 \text{Yield and other buckling} & &= & 24.39 \text{ k} & (\text{E3.1-1})
 \end{aligned}$$

$$\begin{aligned}
 \text{Limit} &= \text{MIN}(\text{MAJOR AND MINOR AXIS}) &= & 22.73 \text{ k} & (\text{E3.1-1})
 \end{aligned}$$

$$\begin{aligned}
 \text{Ratio} &= \text{Axial/Limit} &= & 0.00
 \end{aligned}$$

/*

BENDING STRENGTH (Sections F2, F3, F4, I6)

YIELDING AND GLOBAL (LATERAL TORSIONAL) BUCKLING (Section F2)

$$\begin{aligned}
 \sigma_{ey}, \text{ Euler Buckling Stress} & &= & 53.16 \text{ ksi} & (\text{F2.1.1-4}) \\
 \sigma_t, \text{ Torsional Buckling Stress} & &= & 57.49 \text{ ksi} & (\text{F2.1.1-5}) \\
 F_{cre}, \text{ Elastic Buckling Stress} & &= & 84.31 \text{ ksi} & (\text{F2.1.1-1}) \\
 \text{Since } 2.78 * F_y > F_{cre} > 0.56 * F_y, \\
 F_n &= 10 / 9 * F_y * (1 - (10 * F_y / (36 * F_{cre}))) &= & 50.04 \text{ ksi} & (\text{F2.1-4}) \\
 S_{xx} & &= & 5.116 \text{ in}^3 \\
 M_y &= S_{xx} * F_y &= & 23.45 \text{ f-k} & (\text{F2.1-2}) \\
 M_n &= S_{xx} * F_n \leq M_y &= & 21.33 \text{ f-k} & (\text{F2.1-1}) \\
 \Omega_b & &= & 1.67 \\
 \text{Limit} &= M_n / \Omega_b &= & 12.77 \text{ f-k} & (\text{B3.2.1-2})
 \end{aligned}$$

LOCAL BUCKLING (Section F3)

$$\begin{aligned}
 F_n \text{ (from Section F2)} & &= & 50.04 \text{ ksi} & (\text{F2.1-4}) \\
 &\text{From Effective Section Properties report} \\
 S_{xe} & &= & 4.868 \text{ in}^3 \\
 S_{xet}, \text{ Effective section modulus calculated at extreme fiber tension stress of } F_y \\
 S_{xet} & &= & 4.769 \text{ in}^3 \\
 M_{n1} &= S_{xe} * F_n \leq S_{xet} * F_y &= & 20.30 \text{ f-k} & (\text{F3.1-1}) \\
 \Omega_b & &= & 1.67
 \end{aligned}$$



$$\text{Limit} = M_{n1} / \Omega_b = 12.15 \text{ f-k} \quad (\text{B3.2.1-2})$$

DISTORTIONAL BUCKLING (Section F4 and Appendix 2)

$$\begin{aligned} L_m &= 80.000 \text{ in} \\ L_{crd} &= 21.518 \text{ in} \quad (2.3.3.3-4) \\ L, \text{ smaller of } L_m \text{ and } L_{crd} &= 21.518 \text{ in} \\ \beta &= 1.160 \quad (2.3.3.3-3) \\ k\phi_{fe} &= 1.1611 \text{ k} \quad (2.3.1.3-3) \\ k\phi_{we} &= 1.0590 \text{ k} \quad (2.3.3.3-5) \\ k\phi &= 0.0000 \\ k\phi_{fg} &= 0.03145 \text{ in}^2 \quad (2.3.1.3-5) \\ k\phi_{wg} &= 0.00601 \text{ in}^2 \quad (2.3.3.3-6) \\ \epsilon_{web} &= 2.0000 \\ F_{crd} = \beta * (k\phi_{fe} + k\phi_{we} + k\phi) / (k\phi_{fg} + k\phi_{wg}) &= 68.73 \text{ ksi} \quad (2.3.3.3-2) \\ M_{crd} = S_{xx} * F_{crd} &= 29.30 \text{ f-k} \quad (\text{F4.1-5}) \\ M_y = S_{xx} * F_y &= 23.45 \text{ f-k} \quad (\text{F4.1-4}) \\ \lambda_d = \sqrt{(M_y / M_{crd})} &= 0.89 \quad (\text{F4.1-3}) \\ \text{For } \lambda_d > 0.673, \\ M_{nd} = [1 - 0.22 * (M_{crd} / M_y)^{0.5}] * M_y * (M_{crd} / M_y)^{0.5} &= 19.77 \text{ f-k} \quad (\text{F4.1-2}) \\ \Omega_b &= 1.67 \\ \text{Limit} = M_n / \Omega_b &= 11.84 \text{ f-k} \quad (\text{B3.2.1-2}) \end{aligned}$$

SUMMARY:

COMPRESSION FLANGE

$$\begin{aligned} \text{Yielding and Global Buckling} &= 12.77 \text{ f-k} \quad (\text{F2.1-1}) \\ \text{Local Buckling} &= 12.15 \text{ f-k} \quad (\text{F3.1-1}) \\ \text{Distortional Buckling} &= 11.84 \text{ f-k} \quad (\text{F4.1-2}) \\ \text{Limit} &= 11.84 \text{ f-k} \quad (\text{F4.1-2}) \\ \text{Ratio} &= 0.90 \end{aligned}$$

/*

AXIAL + BENDING STRENGTH

$$\begin{aligned} P, \text{ Required compressive axial strength} &= 0.00 \text{ k} \\ P_a = P_n / \Omega_c, \text{ Available axial strength} &= 22.73 \text{ k} \\ M_x, \text{ Required flexural strength} &= 10.63 \text{ f-k} \\ M_{ax} = M_{nx} / \Omega_b, \text{ Available flexural strength} &= 11.84 \text{ f-k} \end{aligned}$$

COMPRESSION FLANGE

$$\text{Ratio} = P / P_a + M_x / M_{ax} = 0.8981 \quad (\text{H1.2-1})$$

TENSION FLANGE

$$\text{Ratio} = |P / P_a - M_x / M_{ax}| = 0.8122$$

SHEAR & BENDING

$$\begin{aligned} \text{Bending: } M_{no} / \Omega_b &= 13.09 \text{ f-k} \\ \text{Shear: } V_n / \Omega_v &= 10.03 \text{ k} \\ \text{Ratio, shear \& bending:} &= 0.8381 \quad (\text{H2-1}) \end{aligned}$$

WEB CRIPPLING STRENGTH

$$\begin{aligned} R, \text{ inside bend radius} &= 0.188 \text{ in} \\ N, \text{ bearing length} &= 4.000 \text{ in} \\ h, \text{ flat web depth} &= 9.421 \text{ in} \end{aligned}$$

Calcs for bearing loc End

$$\begin{aligned} R/t &= 1.84 < 9.0 \quad (\text{Table G5-2}) \\ C, \text{ general coefficient} &= 4.00 \quad (\text{Table G5-2}) \\ C_r, \text{ radius coefficient} &= 0.14 \quad (\text{Table G5-2}) \\ C_n, \text{ bearing coefficient} &= 0.35 \quad (\text{Table G5-2}) \\ C_h, \text{ web coefficient} &= 0.020 \quad (\text{Table G5-2}) \\ \Omega_w &= 1.75 \quad (\text{Table G5-2}) \\ \theta, \text{ angle between web and lower flange} &= 90.00 \\ P_n, \text{ nominal web crippling strength} &= 4.78 \text{ k} \quad (\text{G5-1}) \\ P_n / \Omega, \text{ allowable bearing force} &= 2.73 \text{ k} \quad (\text{B3.2.1-2}) \end{aligned}$$



BENDING & WEB CRIPPLING

h/t	=	92.36	<= 150
N/t	=	39.22	<= 140
R/t	=	1.84	<= 5.5
F _n , nominal web crippling strength	=	4.78	k
M _{nxo} , nominal flexural strength, stress at F _y	=	21.86	f-k
Bending & web crippling, calculated	=	0.8670	(H3-1a)
Bending & web crippling, limit	=	0.7824	(H3-1a)
Ratio, bending & web crippling	=	1.1082	

/*

AXIAL EFFECTIVE SECTION PROPERTIES

Angle, θ	=	90.00	deg
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MAJOR AXIS

The following sections calculate the effective area for a stress of 32.07 ksi

Top Lip Effective Area: Code 1.2-a

$w = d = D - (t+R) \cdot \tan(\theta/2)$	=	0.461	in
$f_1 = f_2 = F_n$	=	32.07	ksi
$\Psi = f_2/f_1 $	=	1.0000	(1.2.2-1)
K, plate buckling coefficient	=	0.4300	(1.2.2-2)
F _{cr1}	=	562.48	ksi (1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.23877	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
$d_s' = \rho \cdot w$	=	0.461	in (1.1-1)

Check for Lip Stiffener Reduction: Code 1.3

$S = 1.28 \cdot (E/F_n)^{0.5}$	=	38.8229	(1.3-7)
I _a , adequate I of stiffener	=	0.000983	in ⁴ (1.3-8)
I _s , I of stiffener	=	0.000830	in ⁴ (1.3-10)
$R_I = I_s / I_a \leq 1.0$	=	0.8446	(1.3-9)
w, flat flange width	=	2.421	in
w/t > 0.328 S, reduction necessary			
$d_s = d_s' \cdot R_I$	=	0.389	in (1.3-6)
Ineffective width = d-d _s	=	0.072	in (Fig 1.3-1)

Top Flange Effective Area: Code 1.1, 1.3

w, flat width	=	2.421	in (Fig 1.3-1)
$n = 0.582 - (w/t)/(4/S) \geq 1/3$	=	0.429	in (1.3-11)
$k = (4.82 - 5 \cdot D/w) \cdot (R_I)^n + 0.43 \leq 4$	=	3.4724	(Table 1.3-1)
F _{cr1}	=	164.34	ksi (1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.44174	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
b = effective width = w	=	2.421	in (Fig 1.3-1)
Ineffective width	=	0.000	in (Fig 1.3-1)
b ₁ = w/2	=	1.211	in (1.3-2)
b ₂ = w/2	=	1.211	in (1.3-2)

Bottom Lip Effective Area: Code 1.2-a

$w = d = D - (t+R) \cdot \tan(\theta/2)$	=	0.461	in
$f_1 = f_2 = F_n$	=	32.07	ksi
$\Psi = f_2/f_1 $	=	1.0000	(1.2.2-1)
K, plate buckling coefficient	=	0.4300	(1.2.2-2)
F _{cr1}	=	562.48	ksi (1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.23877	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
$d_s' = \rho \cdot w$	=	0.461	in (1.1-1)

Check for Lip Stiffener Reduction: Code 1.3

$S = 1.28 \cdot (E/F_n)^{0.5}$	=	38.8229	(1.3-7)
I _a , adequate I of stiffener	=	0.000983	in ⁴ (1.3-8)
I _s , I of stiffener	=	0.000830	in ⁴ (1.3-10)
$R_I = I_s / I_a \leq 1.0$	=	0.8446	(1.3-9)
w, flat flange width	=	2.421	in
w/t > 0.328 S, reduction necessary			
$d_s = d_s' \cdot R_I$	=	0.389	in (1.3-6)
Ineffective width = d-d _s	=	0.072	in (Fig 1.3-1)

Bottom Flange Effective Area: Code 1.1, 1.3



w, flat width	=	2.421 in	(Fig 1.3-1)
$n = 0.582 - (w/t)/(4/S) \geq 1/3$	=	0.429 in	(1.3-11)
$k = (4.82 - 5*D/w)*(R_t)^n + 0.43 \leq 4$	=	3.4724	(Table 1.3-1)
F_{cr1}	=	164.34 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.44174	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
b = effective width = w	=	2.421 in	(Fig 1.3-1)
/*			
Ineffective width	=	0.000 in	(Fig 1.3-1)
$b_1 = w/2$	=	1.211 in	(1.3-2)
$b_2 = w/2$	=	1.211 in	(1.3-2)
Web Effective Width: Code 1.1.2			
w, flat width	=	9.421 in	(Fig 1.1.2-1)
$\Psi = f_2/f_1 $	=	1.0000	(1.1.2-1)
K, plate buckling coefficient	=	4.0000	(1.1.2-8)
F_{cr1}	=	12.50 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	1.60159	(1.1-3)
$\lambda > 0.673$			
$\rho = (1 - 0.22/\lambda)/\lambda$	=	0.5386	(1.1-2)
b = effective width = $\rho * w$	=	5.074 in	(1.1-1)
Ineffective width	=	4.347 in	(Fig 1.3-1)
$b_1 = b / (3-\Psi)$	=	2.537 in	(1.1.2-9)
$b_2 = b - b_1$	=	2.537 in	(1.1.2-10)
Final results:			
A_e	=	1.2760 in ²	
/*			
AXIAL EFFECTIVE SECTION PROPERTIES			
Angle, θ	=	90.00	deg
MINOR AXIS			
The following sections calculate the effective area for a stress of		35.67 ksi	
Top Lip Effective Area: Code 1.2-a			
$w = d = D - (t+R)*\tan(\theta/2)$	=	0.461 in	
$f_1 = f_2 = F_n$	=	35.67 ksi	
$\Psi = f_2/f_1 $	=	1.0000	(1.2.2-1)
K, plate buckling coefficient	=	0.4300	(1.2.2-2)
F_{cr1}	=	562.48 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.25182	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
$d_s' = \rho * w$	=	0.461 in	(1.1-1)
Check for Lip Stiffener Reduction: Code 1.3			
$S = 1.28*(E/F_n)^{0.5}$	=	36.8105	(1.3-7)
I_a , adequate I of stiffener	=	0.001373 in ⁴	(1.3-8)
I_a , I of stiffener	=	0.000830 in ⁴	(1.3-10)
$R_t = I_s / I_a \leq 1.0$	=	0.6045	(1.3-9)
w, flat flange width	=	2.421 in	
$w/t > 0.328 S$, reduction necessary			
$d_s = d_s' * R_t$	=	0.278 in	(1.3-6)
Ineffective width = $d - d_s$	=	0.182 in	(Fig 1.3-1)
Top Flange Effective Area: Code 1.1, 1.3			
w, flat width	=	2.421 in	(Fig 1.3-1)
$n = 0.582 - (w/t)/(4/S) \geq 1/3$	=	0.421 in	(1.3-11)
$k = (4.82 - 5*D/w)*(R_t)^n + 0.43 \leq 4$	=	3.0767	(Table 1.3-1)
F_{cr1}	=	145.61 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.49494	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
b = effective width = w	=	2.421 in	(Fig 1.3-1)
Ineffective width	=	0.000 in	(Fig 1.3-1)
$b_1 = w/2$	=	1.211 in	(1.3-2)
$b_2 = w/2$	=	1.211 in	(1.3-2)
Bottom Lip Effective Area: Code 1.2-a			
$w = d = D - (t+R)*\tan(\theta/2)$	=	0.461 in	



f1 = f2 = F _a	=	35.67 ksi	
Ψ = f2/f1	=	1.0000	(1.2.2-1)
K, plate buckling coefficient	=	0.4300	(1.2.2-2)
F _{cr1}	=	562.48 ksi	(1.1-4)
λ = (F _a / F _{cr1}) ^{0.5}	=	0.25182	(1.1-3)
λ < 0.673 => fully effective, ρ	=	1.00	(1.1-2)
d _s ' = ρ * w	=	0.461 in	(1.1-1)
Check for Lip Stiffener Reduction: Code 1.3			
S = 1.28*(E/F _a) ^{0.5}	=	36.8105	(1.3-7)
I _a , adequate I of stiffener	=	0.001373 in ⁴	(1.3-8)
I _s , I of stiffener	=	0.000830 in ⁴	(1.3-10)
R _t = I _s / I _a <= 1.0	=	0.6045	(1.3-9)
w, flat flange width	=	2.421 in	
w/t > 0.328 S, reduction necessary			
d _s = d _s ' * R _t	=	0.278 in	(1.3-6)
Ineffective width = d - d _s	=	0.182 in	(Fig 1.3-1)
Bottom Flange Effective Area: Code 1.1, 1.3			
w, flat width	=	2.421 in	(Fig 1.3-1)
n = 0.582 - (w/t)/(4/S) >= 1/3	=	0.421 in	(1.3-11)
k = (4.82 - 5*D/w)*(R _t) ⁿ + 0.43 <= 4	=	3.0767	(Table 1.3-1)
F _{cr1}	=	145.61 ksi	(1.1-4)
λ = (F _a / F _{cr1}) ^{0.5}	=	0.49494	(1.1-3)
λ < 0.673 => fully effective, ρ	=	1.00	(1.1-2)
b = effective width = w	=	2.421 in	(Fig 1.3-1)
/*			
Ineffective width	=	0.000 in	(Fig 1.3-1)
b ₁ = w/2	=	1.211 in	(1.3-2)
b ₂ = w/2	=	1.211 in	(1.3-2)
Web Effective Width: Code 1.1.2			
w, flat width	=	9.421 in	(Fig 1.1.2-1)
Ψ = f2/f1	=	1.0000	(1.1.2-1)
K, plate buckling coefficient	=	4.0000	(1.1.2-8)
F _{cr1}	=	12.50 ksi	(1.1-4)
λ = (F _a / F _{cr1}) ^{0.5}	=	1.68914	(1.1-3)
λ > 0.673			
ρ = (1 - 0.22/λ)/λ	=	0.5149	(1.1-2)
b = effective width = ρ * w	=	4.851 in	(1.1-1)
Ineffective width	=	4.570 in	(Fig 1.3-1)
b ₁ = b / (3-Ψ)	=	2.425 in	(1.1.2-9)
b ₂ = b - b ₁	=	2.425 in	(1.1.2-10)
Final results:			
A _e	=	1.2307 in ²	
/*			
BENDING EFFECTIVE SECTION PROPERTIES			
Angle, θ	=	90.00	deg
Bottom Lip Effective Area: Code 1.2-a			
w = d - (t+R) * tan(θ/2)	=	0.4605 in	
f1 = F _a *(h _o -Ybar-(R+t)*tan(θ/2)*sin(θ))/(h _o -Ybar)	=	47.141 ksi	
f2 = F _a *(h _o -Ybar-D*sin(θ))/(h _o -Ybar)	=	42.532 ksi	
Ψ = f2/f1	=	0.90224	(1.2.2-1)
K = plate buckling coefficient	=	0.46529	(1.2.2-2)
F _{cr1}	=	608.64 ksi	(1.1-4)
λ = (F _a / F _{cr1}) ^{0.5}	=	0.28673	(1.1-3)
λ < 0.673 => fully effective, ds' = w			
f, stress in compression flange	=	50.04 ksi	
S = 1.28*(29500/f) ^{0.5}	=	31.079	(1.3-7)
w, flat flange width	=	2.421 in	
I _a = adequate I of stiffener	=	0.003572 in ⁴	(1.3-8)
I _s = I of stiffener	=	0.000830 in ⁴	(1.3-10)
R _t = I _s /I _a <= 1.0	=	0.2324	(1.3-9)
ds = ds'*R _t	=	0.1070 in	(1.3-6)



Lip ineffective width = d-ds	=	0.3535 in	
$n = [0.582 - (w/t)/4S] \geq 1/3$	=	0.3911	(1.3-11)
K, flange plate buckling coefficient	=	2.2785	(Table 1.3-1)
Flange Effective Area: Code 1.1, 1.3			
F_{cr1} , critical stress	=	107.834 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.6812	(1.1-3)
$\lambda > 0.673 \Rightarrow b_e = \rho * w$			
$\rho = (1 - 0.22/\lambda)/\lambda$	=	0.9939	(1.1-2)
$b_e = \rho * w$	=	2.4062 in	(1.1-1)
Ineffective width = w - $\rho * w$	=	0.0148 in	
Web Effective Width: Code 1.1.2			
w, flat width	=	9.4210 in	
Iteration 1: (After flange and lip reduction)			
$\bar{Y}$ of reduced section = $\bar{Y}_{new}$	=	5.0985 in	
I_x of reduced section	=	24.818 in ⁴	
S_{xe} of reduced section	=	4.868 in ³	
$f1 = F_n * (\bar{Y}_{new} - R - t) / (\bar{Y}_{new})$	=	47.197 ksi	
$f2 = -f1 * (h_o - \bar{Y}_{new} - R - t) / (\bar{Y}_{new} - R - t)$	=	-45.263 ksi	
$\Psi = f2/f1 $	=	0.95904	(1.1.2-1)
K, plate buckling coefficient	=	22.955	(1.1.2-2)
w	=	9.421 in	
w/t	=	92.36	
F_{cr1}	=	71.74 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.81108	(1.1-3)
$\lambda > 0.673 \Rightarrow b_e = \rho * w$			
$\rho = (1 - 0.22/\lambda)/\lambda$	=	0.8985	(1.1-2)
$b_e = \rho * w$	=	8.4648 in	(1.1-1)
$h_o/b_o < 4$, therefore			
b1, width near compression flange	=	2.1381 in	(1.1.2-3)
b2, width near neutral axis	=	4.2324 in	(1.1.2-4)
Compression width available	=	4.8090 in	
$b1 + b2 >$ compression width (fully effective)			
Other flange: No compressive stress (fully effective)			
Other lip: No compressive stress (fully effective)			
Iteration report: No change in web, therefore complete			
Final results:			
S_{xe}	=	4.868 in ³	
/*			
AXIAL EFFECTIVE SECTION PROPERTIES, $F_n = F_y$			
The following sections calculate the effective area for a stress of		55.00 ksi	
Top Lip Effective Area: Code 1.2-a			
$w = d = D - (t+R) * \tan(\theta/2)$	=	0.461 in	
$f1 = f2 = F_n$	=	55.00 ksi	
$\Psi = f2/f1 $	=	1.0000	(1.2.2-1)
K, plate buckling coefficient	=	0.4300	(1.2.2-2)
F_{cr1}	=	562.48 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.31270	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
$d_e' = \rho * w$	=	0.461 in	(1.1-1)
Check for Lip Stiffener Reduction: Code 1.3			
$S = 1.28 * (E/F_n)^{0.5}$	=	29.6442	(1.3-7)
I_a , adequate I of stiffener	=	0.004561 in ⁴	(1.3-8)
I_s , I of stiffener	=	0.000830 in ⁴	(1.3-10)
$R_I = I_s / I_a \leq 1.0$	=	0.1820	(1.3-9)
w, flat flange width	=	2.421 in	
w/t > 0.328 S, reduction necessary			
$d_s = d_s' * R_I$	=	0.084 in	(1.3-6)
Ineffective width = d-ds	=	0.377 in	(Fig 1.3-1)
Top Flange Effective Area: Code 1.1, 1.3			
w, flat width	=	2.421 in	(Fig 1.3-1)
$n = 0.582 - (w/t)/(4/S) \geq 1/3$	=	0.382 in	(1.3-11)



$k = (4.82 - 5 \cdot D/w) \cdot (R_t)^n + 0.43 \leq 4$	=	2.1367	(Table 1.3-1)
F_{cr1}	=	101.12 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.73749	(1.1-3)
$\lambda > 0.673$			
$\rho = (1 - 0.22/\lambda) / \lambda$	=	0.9515	(1.1-2)
$b = \text{effective width} = \rho \cdot w$	=	2.303 in	(1.1-1)
Ineffective width	=	0.118 in	(Fig 1.3-1)
$b_1 = (b/2) \cdot R_t$	=	0.210 in	(1.3-4)
$b_2 = b - b_1$	=	2.094 in	(1.3-5)
Bottom Lip Effective Area: Code 1.2-a			
$w = d = D - (t+R) \cdot \tan(\theta/2)$	=	0.461 in	
$f_1 = f_2 = F_n$	=	55.00 ksi	
$\Psi = f_2/f_1 $	=	1.0000	(1.2.2-1)
K , plate buckling coefficient	=	0.4300	(1.2.2-2)
F_{cr1}	=	562.48 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.31270	(1.1-3)
$\lambda < 0.673 \Rightarrow$ fully effective, ρ	=	1.00	(1.1-2)
$d_s' = \rho \cdot w$	=	0.461 in	(1.1-1)
Check for Lip Stiffener Reduction: Code 1.3			
$S = 1.28 \cdot (E/F_n)^{0.5}$	=	29.6442	(1.3-7)
I_s , adequate I of stiffener	=	0.004561 in ⁴	(1.3-8)
I_s , I of stiffener	=	0.000830 in ⁴	(1.3-10)
$R_t = I_s / I_a \leq 1.0$	=	0.1820	(1.3-9)
w , flat flange width	=	2.421 in	
$w/t > 0.328 S$, reduction necessary			
$d_s = d_s' \cdot R_t$	=	0.084 in	(1.3-6)
Ineffective width = $d - d_s$	=	0.377 in	(Fig 1.3-1)
Bottom Flange Effective Area: Code 1.1, 1.3			
w , flat width	=	2.421 in	(Fig 1.3-1)
$n = 0.582 - (w/t) / (4/S) \geq 1/3$	=	0.382 in	(1.3-11)
$k = (4.82 - 5 \cdot D/w) \cdot (R_t)^n + 0.43 \leq 4$	=	2.1367	(Table 1.3-1)
F_{cr1}	=	101.12 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	0.73749	(1.1-3)
$\lambda > 0.673$			
$\rho = (1 - 0.22/\lambda) / \lambda$	=	0.9515	(1.1-2)
$b = \text{effective width} = \rho \cdot w$	=	2.303 in	(1.1-1)
/*			
Ineffective width	=	0.118 in	(Fig 1.3-1)
$b_1 = (b/2) \cdot R_t$	=	0.210 in	(1.3-4)
$b_2 = b - b_1$	=	2.094 in	(1.3-5)
Web Effective Width: Code 1.1.2			
w , flat width	=	9.421 in	(Fig 1.1.2-1)
$\Psi = f_2/f_1 $	=	1.0000	(1.1.2-1)
K , plate buckling coefficient	=	4.0000	(1.1.2-8)
F_{cr1}	=	12.50 ksi	(1.1-4)
$\lambda = (F_n / F_{cr1})^{0.5}$	=	2.09748	(1.1-3)
$\lambda > 0.673$			
$\rho = (1 - 0.22/\lambda) / \lambda$	=	0.4268	(1.1-2)
$b = \text{effective width} = \rho \cdot w$	=	4.020 in	(1.1-1)
Ineffective width	=	5.401 in	(Fig 1.3-1)
$b_1 = b / (3 - \Psi)$	=	2.010 in	(1.1.2-9)
$b_2 = b - b_1$	=	2.010 in	(1.1.2-10)
Final results:			
A_e	=	1.0823 in ²	



DESIGN OF TOP RAIL: Maximum effective wind area = 36.8 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.90$ $G_{Cpi} = -0.18$ (Figure 30.3-1)
 Zone 4 Leeward : $G_{Cp} = 1.00$ $G_{Cpi} = -0.18$ (leeward controls design)

$$\begin{aligned}\text{Wind pressure} = p &= q_z (G_{Cp} - G_{Cpi}) \\ &= 86.0 (1.00 - (-0.18)) \\ &= 101.5 \text{ lbs/sq. ft.}\end{aligned}$$

$$\begin{aligned}\text{Wind Pressure (ASD)} &= 101.5 (0.6) \\ &= 60.9 \text{ psf}\end{aligned}$$

$$\begin{aligned}\text{Unit load calc} &= (60.9 \text{ lb/sq.ft.})(1.75 \text{ ft.}) \\ &= 106.6 \text{ lb/ft} = .0089 \text{ k/in}\end{aligned}$$

$$\begin{aligned}\text{Deflection calculation} &= 5wl^4 / 384 EI \\ &= 5 (0.0089)(252)^4 / (384)(29000)(67.4) \\ &= 0.24 \text{ in} < 2.10 \text{ in (L/120) OK!!!}\end{aligned}$$

Design Check for Top Rail:

STRUCTURAL CODE

Design Basis - Working Stress (WS)
 Hot Rolled Steel - AISC16

MEMBER

Member Part - C10X15.3

Web Depth (in) - 9.130 UNITS (k , f-k)

Web Thick (in) - 0.240 ACTION CALC LIMIT RATIO EQUATION

Flange Width (in) - 2.600 ----- ----- ----- ----- -----

Top Flange Thick(in)- 0.436 Axial 3.00 48.28 0.06 E7-1

Bot Flange Thick(in)- 0.436

Yield Flange (ksi) - 50.000 Shear 1.12 43.12 0.03 G2-1

Yield Web (ksi) - 50.000

Stiffener Space(in) - 0.000 Bending_Top 5.88 39.53 0.15 F2-1

UNBRACED LENGTH (Tension)

Major Axis (ft) - 21.000

Minor Axis Axial(ft)- 7.000 Bending_Bot 5.88 27.05 0.22 F2-2

Minor Axis Top (ft) - 7.000 (Compression)

Minor Axis Bot (ft) - 7.000

Cb Top - 1.000 Axial+Bending_Top 0.13

Cb Bottom - 1.000

CONSTANTS Axial+Bending_Bot 0.23 H1-1b

Tension Coefficient - 1.000

ACTION **MEMBER CLASSIFICATION**

Axial (k) - 3.000 Axial : Web = Slender , Flange = Non-Slender

Shear (k) - 1.120 Bending: Web = Compact , Flange = Compact

Moment (f-k) - 5.880

SLENDERNESS

Major Axis 65.12

Minor Axis 117.81

/*



CALCULATIONS

DESIGN BASIS = WORKING STRESS (ASD)

SECTION SIZE

Member Part	=	C10X15.3
d, Total Depth	=	10.002 in
h _w , Web Depth	=	9.130 in
t _w , Web Thickness	=	0.240 in
b _f , Flange Width	=	2.600 in
t _{ft} , Top Flange Thickness	=	0.436 in
t _{fb} , Bottom Flange Thickness	=	0.436 in
Stiffener Use	=	N
a, Stiffener Spacing	=	0.000 in

MATERIAL

F _{yw} , Web Yield Stress	=	50.00 ksi
F _{uw} , Web Ultimate Stress	=	65.00 ksi
F _{yf} , Flange Yield Stress	=	50.00 ksi
F _{uf} , Flange Ultimate Stress	=	65.00 ksi

CONSTANTS

E, Modulus of Elasticity	=	29000.00 ksi
G, Shear Modulus	=	11200.00 ksi
Ω _v , Safety Factor for Shear	=	1.67
Ω _c , Safety Factor for Axial Compression	=	1.67
Ω _b , Safety Factor for Bending	=	1.67
C _n = A _e /A _g , Tension Net Area Coefficient	=	1.00

UNBRACED LENGTHS

L _x , Major Axis	=	21.0000 ft
L _y , Minor Axis (Axial)	=	7.0000 ft
L _z , Minor Axis (Axial)	=	7.0000 ft
L _{bt} , Minor Axis, Top Flange	=	7.0000 ft
L _{bb} , Minor Axis, Bottom Flange	=	7.0000 ft
C _b , Bending Coefficient	=	1.00

LOADING - WORKING LOADS

P _x , Axial (Compression)	=	3.00 k
V _x , Shear	=	1.12 k
M _x , Moment (major axis, bottom flange in compression)	=	5.88 f-k

MEMBER CLASSIFICATION

AXIAL

e = √(E/F _{yw})	=	24.083
λ _{pw} = Not applicable		
λ _{rw} = 1.49*e	=	35.88
λ _w = h _w /t _w = 38.0 > 35.9	Web is Slender	
λ _{pf} = Not applicable		
K _c = 4/√(h _w /t _w) ≥ 0.35, ≤ 0.76		0.6485
λ _{rf} = 0.64*√(K _c *E/F _{yf})		12.41
λ _{fo} = b _{fo} /t _{fo} = 5.96 < 12.41	Top Flange is Non-Slender	
λ _{fi} = b _{fi} /t _{fi} = 5.96 < 12.41	Bottom Flange is Non-Slender	

BENDING

e = √(E/F _{yw})	=	24.083
/*		
λ _{pw} = 3.76*e,	Compact limit	90.553
λ _{rw} = 5.70*e,	Non-compact limit	137.274
λ _w = h _w /t _w = 38.0 < 90.6	Web is Compact	
λ _{pf} = 0.38*√(E/F _{yf}),	Compact limit	9.152
K _c = 4/√(h _w /t _w) ≥ 0.35, ≤ 0.76		0.649
S _{xt} /S _{xc}		1.000



Since $S_{xt}/S_{xc} = 1.000 \geq 0.7$,
 $F_L = 0.7 \cdot F_{yf}$ = 35.000 ksi
 $\lambda_{xf} = 0.95 \cdot \sqrt{(K_c \cdot E / F_L)}$, Non-compact limit = 22.022
 $\lambda_{x0} = b_{x0} / t_{x0} = 5.96 < 9.15$ Top Flange is Compact
 $\lambda_{x1} = b_{x1} / t_{x1} = 5.96 < 9.15$ Bottom Flange is Compact

SECTION PROPERTIES

d , overall member depth = 10.002 in
 h_o , distance between flange centroids = 9.566 in
 A_g , gross area = 4.490 in²
 Y_{bar} , centroid, from bottom = 5.001 in
 I_x , major axis moment of inertia = 67.4000 in⁴
 S_{xt} , elastic section modulus, tension flange = 13.500 in³
 S_{xc} , elastic section modulus, compression flange = 13.500 in³
 r_x , major axis radius of gyration = 3.870 in
 I_y , minor axis moment of inertia = 2.2800 in⁴
 S_y , elastic section modulus for y-axis = 1.160 in³
 r_y , minor axis radius of gyration = 0.713 in
 Z_x , plastic section modulus = 15.845 in³
 J , Torsional Constant = 0.181111 in⁴
 r_t , torsion radius of gyration = 0.654 in
 I_{yc} , minor axis moment of inertia in compression = 0.644 in⁴
 I_{yt} , minor axis moment of inertia in tension = 0.644 in⁴
 C_w , warping constant = 45.240 in⁶
 x_o , shear center with respect to centroid of section = -0.916 in
 y_o , shear center with respect to centroid of section = 0.000 in
 r_o , polar radius of gyration about shear center = 3.917 in
 β_x , asymmetry parameter = -3.743 in

SHEAR STRENGTH

UNSTIFFENED WEB, (Chapter G2.1)

k_v = 5.3400
 $h_w/t_w = 38.04$
 $1.10 \cdot \sqrt{(k_v \cdot E / F_{yw})}$ = 61.22
 Since $h_w/t_w \leq 1.10 \cdot \sqrt{(k_v \cdot E / F_{yw})}$,
 C_{v1} = 1.000 (G2-3)
 $V_n = 0.60 \cdot F_{yw} \cdot d \cdot t_w \cdot C_{v1}$ = 72.014 k (G2-)
 1) Limit = V_n / Ω_v = 43.122 k
 Ratio = $V_x \cdot \Omega_v / V_n$ = 0.0260

AXIAL STRENGTH

COMPRESSION OF SLENDER ELEMENTS, (Chapter E, E7)

Calculate F_e

L_{cx} , major axis length = $K \cdot L_x$ = 21.000 ft
 L_{cy} , minor axis length = $K \cdot L_y$ = 7.000 ft
 L_{cz} , longitudinal axis length = $K \cdot L_z$ = 7.000 ft
 L_{cx}/r_x major = 65.116
 L_{cy}/r_y minor = 117.812

F_e for flexural buckling:

$F_{ex} = \pi^2 \cdot E / (L_{cx}/r_x)^2$ (in-plane, major axis) = 67.50 ksi (E3-4)

$F_{ey} = \pi^2 \cdot E / (L_{cy}/r_y)^2$ (out-of-plane, minor axis) = 20.62 ksi (E3-4)

F_{ez} for torsional or flexural-torsional buckling:

$F_{ez} = [(\pi^2 \cdot E \cdot C_w / L_{cz}^2) + G \cdot J] \cdot (1 / (I_x + I_y))$ = 55.45 ksi (E4-2)

/*

Use the minimum F_e = 20.62 ksi

Calculate P_n

$4.71 \cdot \sqrt{[E / (F_{yf})]}$ = 113.43

Since maximum $L_c/r > 4.71 \cdot \sqrt{[E / (F_{yf})]}$

$F_{cr} = 0.877 \cdot F_e$ = 18.08 ksi (E3-3)



$$\begin{aligned}
 A_e \text{ (from Annex A)} &= 4.458 \text{ in}^2 \\
 P_n = A_e \cdot F_{cr} &= 80.630 \text{ k} \quad (\text{E7-1}) \\
 \text{Limit, } P_c = P_n / \Omega &= 48.281 \text{ k} \\
 \text{Ratio} = P_x / P_c &= 0.0621
 \end{aligned}$$

/*

BENDING STRENGTH

COMPRESSION FLANGE (Chapter F, F2)

$$\begin{aligned}
 \text{Flange in compression} &= \text{Bottom} \\
 h_c &= 9.130 \text{ in} \\
 h_p &= 9.130 \text{ in} \\
 \text{FLANGE YIELDING} & \\
 M_n = M_p = F_{yt} \cdot Z_x &= 66.023 \text{ f-k} \quad (\text{F2-1})
 \end{aligned}$$

LATERAL-TORSIONAL BUCKLING

$$\begin{aligned}
 r_{ts} &= \sqrt{\sqrt{I_y \cdot C_w} / S_x} = 0.867 \text{ in} \quad (\text{F2-7}) \\
 L_p &= 1.76 \cdot r_{ts} \cdot \sqrt{E / F_{yt}} = 30.222 \text{ in} \quad (\text{F2-5}) \\
 \text{Member type is U, therefore} & \\
 C &= h_o / 2 \cdot \sqrt{I_y / C_w} = 1.074 \\
 C_2 &= J \cdot C / (S_{xc} \cdot h_o) = 0.001506 \\
 C_1 &= \sqrt{C_2 \cdot \sqrt{[1 + \sqrt{1 + 6.76 \cdot (0.7 \cdot F_{yt} / E / C_2)^2}]}} = 0.0706 \\
 L_r &= 1.95 \cdot r_{ts} \cdot E / (0.7 \cdot F_{yt}) \cdot C_1 = 98.959 \text{ in} \quad (\text{F2-6}) \\
 \text{Since } L_p < L_b \leq L_r, \text{ Use this formula for } M_n & \\
 M_p = Z_x \cdot F_{yt} &= 66.023 \text{ f-k} \\
 M_n = C_b \cdot [M_p - (M_p - 0.7 \cdot F_{yt} \cdot S_x) \cdot (L_b - L_p) / (L_r - L_p)] \leq M_p &= 45.174 \text{ f-k} \quad (\text{F2-2})
 \end{aligned}$$

NOMINAL STRENGTH

$$\begin{aligned}
 M_n \text{ is the minimum of Flange Yielding and Lateral-Torsional Buckling} & \\
 M_n &= 45.174 \text{ f-k} \\
 \text{Limit} = M_n / \Omega_b &= 27.050 \text{ f-k} \\
 \text{Ratio} = M_x \cdot \Omega_b / M_n &= 0.217
 \end{aligned}$$

TENSION FLANGE YIELDING, (Chapter F, F2)

$$\begin{aligned}
 \text{Flange in tension} &= \text{Top} \\
 \text{Since } S_{xt} \geq S_{xc}, \text{ the limit state of tension flange yielding does not apply} & \\
 M_n = F_{yt} \cdot Z_x &= 66.023 \text{ f-k} \quad (\text{F2-1}) \\
 \text{Limit} = M_n / \Omega_b &= 39.535 \text{ f-k} \\
 \text{Ratio} = M_x \cdot \Omega / M_n &= 0.149
 \end{aligned}$$

AXIAL + BENDING STRENGTH (Chapter H)

$$\begin{aligned}
 P_x, \text{ required axial strength} &= 3.000 \text{ k} \\
 P_c, \text{ minimum compressive strength} &= 48.281 \text{ k} \\
 M_x, \text{ required bending strength} &= 5.880 \text{ f-k} \\
 M_c = M_n / \Omega_b, \text{ allowable bending strength, compression flange} &= 27.050 \text{ f-k} \\
 M_{ct} = M_n / \Omega_b, \text{ allowable bending strength, tension flange} &= 39.535 \text{ f-k} \\
 A_e, \text{ effective area} &= 4.458 \text{ in}^2
 \end{aligned}$$

COMPRESSION FLANGE

H1-1 FORMULA

$$\begin{aligned}
 \text{For } P_x / P_c < 0.20, & \\
 \text{Ratio} = P_x / (2 \cdot P_c) + M_x / M_c &= 0.2484 \quad (\text{H1-1b})
 \end{aligned}$$

H1-2 FORMULAS

Since this is a rolled compact double symmetric section and single axis flexure, we may use Section H1.3 in lieu of the H1-1 formulas.

OUT-OF-PLANE BUCKLING

$$\text{Ratio} = (P_x / P_{co}) \cdot (1.5 - 0.5 \cdot P_x / P_{co}) + (M_x / M_{cx})^2 = 0.1379 \quad (\text{H1-2})$$

IN-PLANE INSTABILITY

$$\begin{aligned}
 \text{For } P_x / P_{ci} < 0.20, & \\
 \text{Ratio} = P_x / (2 \cdot P_{ci}) + M_x / M_c &= 0.2288 \quad (\text{H1-1b})
 \end{aligned}$$

MAXIMUM RATIO OF H1-2 FORMULAS

$$\text{Ratio} = 0.2288$$

MINIMUM RATIO OF H1-1 and H1-2 FORMULAS

$$\begin{aligned}
 \text{H1-2 formula controls} & \\
 \text{Ratio} &= 0.2288
 \end{aligned}$$

TENSION FLANGE



$f_a = P_x/A_e$, Axial Stress	=	0.67 ksi
/*		
$f_b = M_x/S_{xt}$, Bending tension stress	=	5.23 ksi
$f_a \leq f_b$, therefore:		
Ratio = $(M_x/M_{ct}) - (P_x/A_e) / (M_{ct}/S_{xt})$	=	0.1296
/*		
ANNEX A		
EFFECTIVE AREA FOR SLENDER MEMBERS		
AXIAL, (Chapter E7)		
Use the minimum F_e	=	20.62 ksi
$F_{cr} = 0.877 * F_e$	=	18.08 ksi (E3-3)
Web		
$\lambda = h_w/t_w$	=	38.04
$\lambda_x = 1.49 * \sqrt{(E/F_{yw})}$	=	35.88
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	59.67
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$		
$h_e = h_w$	=	9.130 in (E7-2)
t	=	0.240 in
$A1 = h_e * t$	=	2.191 in ²
Top flange		
$\lambda = b/t$	=	2.98
$\lambda_x = 0.56 * \sqrt{(E/F_y)}$	=	13.49
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	22.42
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$		
$b_e = b$	=	1.300 in (E7-2)
t	=	0.436 in
$A2 = (b_e * t) * 2$	=	1.134 in ²
Bottom flange		
$\lambda = b/t$	=	2.98
$\lambda_x = 0.56 * \sqrt{(E/F_y)}$	=	13.49
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	22.42
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$		
$b_e = b$	=	1.300 in (E7-2)
t	=	0.436 in
$A3 = (b_e * t) * 2$	=	1.134 in ²
Effective Area		
$A_e = A1 + A2 + A3$	=	4.458 in ²



DESIGN OF BOTTOM RAIL: Maximum effective wind area = 36.8 sq. ft.

Zone 4 Windward: $GC_p = 0.90$ $GC_{pi} = -0.18$ (Figure 30.3-1)
 Zone 4 Leeward : $GC_p = 1.00$ $GC_{pi} = -0.18$ (leeward controls design)

$$\begin{aligned}\text{Wind pressure} &= p = q_z (GC_p - GC_{pi}) \\ &= 86.0 (1.00 - (-0.18)) \\ &= 101.5 \text{ lbs/sq. ft.}\end{aligned}$$

$$\begin{aligned}\text{Wind Pressure (ASD)} &= 101.5 (0.6) \\ &= 60.9 \text{ psf}\end{aligned}$$

$$\begin{aligned}\text{Unit load calc} &= (60.9 \text{ lb/sq.ft.})(1.75 \text{ ft.}) \\ &= 106.6 \text{ lb/ft} = .0089 \text{ k/in}\end{aligned}$$

$$\begin{aligned}\text{Deflection calculation} &= 5wl^4 / 384 EI \\ &= 5 (0.0089)(252)^4 / (384)(29000)(67.4) \\ &= 0.24 \text{ in} < 2.10 \text{ in (L/120) OK!!!}\end{aligned}$$

Design Check for Bottom Rail:

STRUCTURAL CODE

Design Basis - Working Stress (WS)
 Hot Rolled Steel - AISC16

MEMBER

Member Part - C10X15.3

Web Depth (in) - 9.130

Web Thick (in) - 0.240

Flange Width (in) - 2.600

Top Flange Thick(in)- 0.436

Bot Flange Thick(in)- 0.436

Yield Flange (ksi) - 50.000

Yield Web (ksi) - 50.000

Stiffener Space(in) - 0.000

UNBRACED LENGTH

Major Axis (ft) - 21.000

Minor Axis Axial(ft)- 7.000

Minor Axis Top (ft) - 7.000

Minor Axis Bot (ft) - 7.000

Cb Top - 1.000

Cb Bottom - 1.000

CONSTANTS

Tension Coefficient - 1.000

ACTION

Axial (k) - 3.000

Shear (k) - 1.120

Moment (f-k) - 5.880

		UNITS(k , f-k)			
ACTION	CALC	LIMIT	RATIO	EQUATION	
-----	-----	-----	-----	-----	
Axial	3.00	48.28	0.06	E7-1	
Shear	1.12	43.12	0.03	G2-1	
Bending_Top (Tension)	5.88	39.53	0.15	F2-1	
Bending_Bot (Compression)	5.88	27.05	0.22	F2-2	
Axial+Bending_Top			0.13		
Axial+Bending Bot			0.23	H1-1b	

MEMBER CLASSIFICATION

Axial : Web = Slender , Flange = Non-Slender
 Bending: Web = Compact , Flange = Compact

SLENDERNESS

Major Axis 65.12
 Minor Axis 117.81

/*



CALCULATIONS

DESIGN BASIS = WORKING STRESS (ASD)

SECTION SIZE

Member Part	=	C10X15.3
d, Total Depth	=	10.002 in
h _w , Web Depth	=	9.130 in
t _w , Web Thickness	=	0.240 in
b _f , Flange Width	=	2.600 in
t _{ft} , Top Flange Thickness	=	0.436 in
t _{fo} , Bottom Flange Thickness	=	0.436 in
Stiffener Use	=	N
a, Stiffener Spacing	=	0.000 in

MATERIAL

F _{yw} , Web Yield Stress	=	50.00 ksi
F _{uw} , Web Ultimate Stress	=	65.00 ksi
F _{ye} , Flange Yield Stress	=	50.00 ksi
F _{ut} , Flange Ultimate Stress	=	65.00 ksi

CONSTANTS

E, Modulus of Elasticity	=	29000.00 ksi
G, Shear Modulus	=	11200.00 ksi
Ω _v , Safety Factor for Shear	=	1.67
Ω _c , Safety Factor for Axial Compression	=	1.67
Ω _b , Safety Factor for Bending	=	1.67
C _n = A _n /A _g , Tension Net Area Coefficient	=	1.00

UNBRACED LENGTHS

L _x , Major Axis	=	21.0000 ft
L _y , Minor Axis (Axial)	=	7.0000 ft
L _z , Minor Axis (Axial)	=	7.0000 ft
L _{bt} , Minor Axis, Top Flange	=	7.0000 ft
L _{bo} , Minor Axis, Bottom Flange	=	7.0000 ft
C _b , Bending Coefficient	=	1.00

LOADING - WORKING LOADS

P _x , Axial (Compression)	=	3.00 k
V _x , Shear	=	1.12 k
M _x , Moment (major axis, bottom flange in compression)	=	5.88 f-k

MEMBER CLASSIFICATION

AXIAL

e = √(E/F _{yw})	=	24.083
λ _{pw} = Not applicable		
λ _{rw} = 1.49*e	=	35.88
λ _w = h _w /t _w = 38.0 > 35.9	Web is Slender	
λ _{pf} = Not applicable		
K _c = 4/√(h _w /t _w) ≥ 0.35, ≤ 0.76		0.6485
λ _{rf} = 0.64*√(K _c *E/F _{ye})		12.41
λ _{fo} = b _{fo} /t _{fo} = 5.96 < 12.41	Top Flange is Non-Slender	
λ _{fi} = b _{fi} /t _{fi} = 5.96 < 12.41	Bottom Flange is Non-Slender	

BENDING

e = √(E/F _{yw})	=	24.083
/*		
λ _{pw} = 3.76*e,	Compact limit	90.553
λ _{rw} = 5.70*e,	Non-compact limit	137.274
λ _w = h _w /t _w = 38.0 < 90.6	Web is Compact	
λ _{pf} = 0.38*√(E/F _{ye}),	Compact limit	9.152
K _c = 4/√(h _w /t _w) ≥ 0.35, ≤ 0.76		0.649
S _{xt} /S _{xc}		1.000



Since $S_{xt}/S_{xc} = 1.000 \geq 0.7$,
 $F_L = 0.7 \cdot F_{yf}$ = 35.000 ksi
 $\lambda_{xf} = 0.95 \cdot \sqrt{(K_c \cdot E / F_L)}$, Non-compact limit = 22.022
 $\lambda_{fo} = b_{fo} / t_{fo} = 5.96 < 9.15$ Top Flange is Compact
 $\lambda_{fi} = b_{fi} / t_{fi} = 5.96 < 9.15$ Bottom Flange is Compact

SECTION PROPERTIES

d , overall member depth = 10.002 in
 h_o , distance between flange centroids = 9.566 in
 A_g , gross area = 4.490 in²
 Y_{bar} , centroid, from bottom = 5.001 in
 I_x , major axis moment of inertia = 67.4000 in⁴
 S_{xt} , elastic section modulus, tension flange = 13.500 in³
 S_{xc} , elastic section modulus, compression flange = 13.500 in³
 r_x , major axis radius of gyration = 3.870 in
 I_y , minor axis moment of inertia = 2.2800 in⁴
 S_y , elastic section modulus for y-axis = 1.160 in³
 r_y , minor axis radius of gyration = 0.713 in
 Z_x , plastic section modulus = 15.845 in³
 J , Torsional Constant = 0.181111 in⁴
 r_t , torsion radius of gyration = 0.654 in
 I_{yc} , minor axis moment of inertia in compression = 0.644 in⁴
 I_{yt} , minor axis moment of inertia in tension = 0.644 in⁴
 C_w , warping constant = 45.240 in⁶
 x_o , shear center with respect to centroid of section = -0.916 in
 y_o , shear center with respect to centroid of section = 0.000 in
 r_o , polar radius of gyration about shear center = 3.917 in
 β_x , asymmetry parameter = -3.743 in

SHEAR STRENGTH

UNSTIFFENED WEB, (Chapter G2.1)
 k_v = 5.3400
 $h_w / t_w = 38.04$
 $1.10 \cdot \sqrt{(k_v \cdot E / F_{yw})}$ = 61.22
 Since $h_w / t_w \leq 1.10 \cdot \sqrt{(k_v \cdot E / F_{yw})}$,
 C_{v1} = 1.000 (G2-3)
 $V_n = 0.60 \cdot F_{yw} \cdot d \cdot t_w \cdot C_{v1}$ = 72.014 k (G2-)
 1) Limit = V_n / Ω_v = 43.122 k
 Ratio = $V_x \cdot \Omega_v / V_n$ = 0.0260

AXIAL STRENGTH

COMPRESSION OF SLENDER ELEMENTS, (Chapter E, E7)
 Calculate F_e
 L_{cx} , major axis length = $K \cdot L_x$ = 21.000 ft
 L_{cy} , minor axis length = $K \cdot L_y$ = 7.000 ft
 L_{cz} , longitudinal axis length = $K \cdot L_z$ = 7.000 ft
 L_{cx} / r_x major = 65.116
 L_{cy} / r_y minor = 117.812
 F_e for flexural buckling:
 $F_{ex} = \pi^2 \cdot E / (L_{cx} / r_x)^2$ (in-plane, major axis) = 67.50 ksi (E3-4)
 $F_{ey} = \pi^2 \cdot E / (L_{cy} / r_y)^2$ (out-of-plane, minor axis) = 20.62 ksi (E3-4)
 F_{ez} for torsional or flexural-torsional buckling:
 $F_{ez} = [(\pi^2 \cdot E \cdot C_w / L_{cz}^2) + G \cdot J] \cdot (1 / (I_x + I_y))$ = 55.45 ksi (E4-2)
 /* Use the minimum F_e = 20.62 ksi
 Calculate P_n
 $4.71 \cdot \sqrt{E / (F_{yf})}$ = 113.43
 Since maximum $L_c / r > 4.71 \cdot \sqrt{E / (F_{yf})}$
 $F_{cr} = 0.877 \cdot F_e$ = 18.08 ksi (E3-3)



$$\begin{aligned}
 A_e \text{ (from Annex A)} &= 4.458 \text{ in}^2 \\
 P_n &= A_e \cdot F_{cr} = 80.630 \text{ k} \quad (\text{E7-1}) \\
 \text{Limit, } P_c &= P_n / \Omega = 48.281 \text{ k} \\
 \text{Ratio} &= P_r / P_c = 0.0621
 \end{aligned}$$

/*

BENDING STRENGTH

COMPRESSION FLANGE (Chapter F, F2)

$$\begin{aligned}
 \text{Flange in compression} &= \text{Bottom} \\
 h_c &= 9.130 \text{ in} \\
 h_p &= 9.130 \text{ in} \\
 \text{FLANGE YIELDING} & \\
 M_n &= M_p = F_{yt} \cdot Z_x = 66.023 \text{ f-k} \quad (\text{F2-1})
 \end{aligned}$$

LATERAL-TORSIONAL BUCKLING

$$\begin{aligned}
 r_{ts} &= \sqrt{I_y \cdot C_w / S_x} = 0.867 \text{ in} \quad (\text{F2-7}) \\
 L_p &= 1.76 \cdot r_{ts} \cdot \sqrt{E / F_{yt}} = 30.222 \text{ in} \quad (\text{F2-5}) \\
 \text{Member type is U, therefore} & \\
 C &= h_o / 2 \cdot \sqrt{I_y / C_w} = 1.074 \\
 C2 &= J \cdot C / (S_{xc} \cdot h_o) = 0.001506 \\
 C1 &= \sqrt{C2 \cdot [1 + \sqrt{1 + 6.76 \cdot (0.7 \cdot F_{yt} / E / C2)^2}]} = 0.0706 \\
 L_r &= 1.95 \cdot r_{ts} \cdot \sqrt{E / (0.7 \cdot F_{yt})} \cdot C1 = 98.959 \text{ in} \quad (\text{F2-6}) \\
 \text{Since } L_p < L_b \leq L_r, \text{ Use this formula for } M_n & \\
 M_p &= Z_x \cdot F_{yt} = 66.023 \text{ f-k} \\
 M_n &= C_b \cdot [M_p - (M_p - 0.7 \cdot F_{yt} \cdot S_x) \cdot (L_b - L_p) / (L_r - L_p)] \leq M_p = 45.174 \text{ f-k} \quad (\text{F2-2})
 \end{aligned}$$

NOMINAL STRENGTH

$$\begin{aligned}
 M_n &\text{ is the minimum of Flange Yielding and Lateral-Torsional Buckling} \\
 M_n &= 45.174 \text{ f-k} \\
 \text{Limit} &= M_n / \Omega_b = 27.050 \text{ f-k} \\
 \text{Ratio} &= M_r \cdot \Omega_b / M_n = 0.217
 \end{aligned}$$

TENSION FLANGE YIELDING, (Chapter F, F2)

$$\begin{aligned}
 \text{Flange in tension} &= \text{Top} \\
 \text{Since } S_{xt} \geq S_{xc}, \text{ the limit state of tension flange yielding does not apply} & \\
 M_n &= F_{yt} \cdot Z_x = 66.023 \text{ f-k} \quad (\text{F2-1}) \\
 \text{Limit} &= M_n / \Omega_b = 39.535 \text{ f-k} \\
 \text{Ratio} &= M_r \cdot \Omega_b / M_n = 0.149
 \end{aligned}$$

AXIAL + BENDING STRENGTH (Chapter H)

$$\begin{aligned}
 P_r, \text{ required axial strength} &= 3.000 \text{ k} \\
 P_c, \text{ minimum compressive strength} &= 48.281 \text{ k} \\
 M_r, \text{ required bending strength} &= 5.880 \text{ f-k} \\
 M_c = M_n / \Omega_b, \text{ allowable bending strength, compression flange} &= 27.050 \text{ f-k} \\
 M_{ct} = M_n / \Omega_b, \text{ allowable bending strength, tension flange} &= 39.535 \text{ f-k} \\
 A_e, \text{ effective area} &= 4.458 \text{ in}^2
 \end{aligned}$$

COMPRESSION FLANGE

H1-1 FORMULA

$$\begin{aligned}
 \text{For } P_r / P_c < 0.20, \\
 \text{Ratio} &= P_r / (2 \cdot P_c) + M_r / M_c = 0.2484 \quad (\text{H1-1b})
 \end{aligned}$$

H1-2 FORMULAS

Since this is a rolled compact double symmetric section and single axis flexure, we may use Section H1.3 in lieu of the H1-1 formulas.

OUT-OF-PLANE BUCKLING

$$\text{Ratio} = (P_r / P_c) \cdot (1.5 - 0.5 \cdot P_r / P_c) + (M_r / M_{cx})^2 = 0.1379 \quad (\text{H1-2})$$

IN-PLANE INSTABILITY

$$\begin{aligned}
 \text{For } P_r / P_{c1} < 0.20, \\
 \text{Ratio} &= P_r / (2 \cdot P_{c1}) + M_r / M_c = 0.2288 \quad (\text{H1-1b})
 \end{aligned}$$

MAXIMUM RATIO OF H1-2 FORMULAS

$$\text{Ratio} = 0.2288$$

MINIMUM RATIO OF H1-1 and H1-2 FORMULAS

$$\begin{aligned}
 \text{H1-2 formula controls} & \\
 \text{Ratio} &= 0.2288
 \end{aligned}$$

TENSION FLANGE



$f_a = P_x/A_e$, Axial Stress	=	0.67 ksi	
/*			
$f_b = M_x/S_{xt}$, Bending tension stress	=	5.23 ksi	
$f_a \leq f_b$, therefore:			
Ratio = $(M_x/M_{ct}) - (P_x/A_e) / (M_{ct}/S_{xt})$	=	0.1296	
/*			
ANNEX A			
EFFECTIVE AREA FOR SLENDER MEMBERS			
AXIAL, (Chapter E7)			
Use the minimum F_e	=	20.62 ksi	
$F_{cr} = 0.877 * F_e$	=	18.08 ksi (E3-3)	
Web			
$\lambda = h_w/t_w$	=	38.04	
$\lambda_x = 1.49 * \sqrt{(E/F_y)}$	=	35.88	
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	59.67	
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$			
$h_e = h_w$	=	9.130 in	(E7-2)
t	=	0.240 in	
$A1 = h_e * t$	=	2.191 in ²	
Top flange			
$\lambda = b/t$	=	2.98	
$\lambda_x = 0.56 * \sqrt{(E/F_y)}$	=	13.49	
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	22.42	
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$			
$b_e = b$	=	1.300 in	(E7-2)
t	=	0.436 in	
$A2 = (b_e * t) * 2$	=	1.134 in ²	
Bottom flange			
$\lambda = b/t$	=	2.98	
$\lambda_x = 0.56 * \sqrt{(E/F_y)}$	=	13.49	
$\lambda_x * \sqrt{(F_y/F_{cr})}$	=	22.42	
Since $\lambda < \lambda_x * \sqrt{(F_y/F_{cr})}$			
$b_e = b$	=	1.300 in	(E7-2)
t	=	0.436 in	
$A3 = (b_e * t) * 2$	=	1.134 in ²	
Effective Area			
$A_e = A1 + A2 + A3$	=	4.458 in ²	

**LOADS AND REACTIONS – SLIDER DOOR ASSEMBLY****NOTES:**

- 1.) VERTICAL REACTION ON SLIDER DOOR ASSEMBLY IS DUE TO DEAD WIEGHT OF DOOR. AT THE MAXIMUM DOOR DIMENSIONS THE LOAD IS 3.0 KIPS AT EACH WHEEL.
- 2.) DOORS ARE MANUALLY OPENED.

GUIDE WHEELS AT STRUCTURAL HEADER

Max. wind area = 173.3 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.76$ $G_{Cpi} = -0.18$
Zone 4 Leeward : $G_{Cp} = 0.86$ $G_{Cpi} = -0.18$ (leeward controls)

(Figure 30.3-1)

Wind pressure (ASD) $p = q_z (G_{Cp} - G_{Cpi}) (0.6)$
 $= 86.0 (.86 - (-0.18)) (0.6)$
 $= 53.7 \text{ lbs/sq. ft.}$

Reaction at guide wheels = $(53.70 \text{ lbs/sq.ft.})(173.3 \text{ sq. ft.})$
 $= 9.3 \text{ kips (horizontal)}$

ANCHOR PIN IN CONCRETE (6" EMBEDMENT)

Max. wind area = 36.8 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.90$ $G_{Cpi} = -0.18$
Zone 4 Leeward : $G_{Cp} = 1.00$ $G_{Cpi} = -0.18$ (leeward controls)

(Figure 30.3-1)

Wind pressure (ASD) $p = q_z (G_{Cp} - G_{Cpi}) (0.6)$
 $= 86.0 (1.00 - (-0.18)) (0.6)$
 $= 60.9 \text{ lbs/sq. ft.}$

Reaction at anchor pin = $(60.9 \text{ lbs/sq.ft.})(36.8 \text{ sq. ft.})$
 $= 2.24 \text{ kips (horizontal)}$

SIDE RAIL AT BUILDING COLUMN

Max. wind area = 210sq. ft.

Zone 4 Windward: $G_{Cp} = 0.78$ $G_{Cpi} = -0.18$
Zone 4 Leeward : $G_{Cp} = 0.87$ $G_{Cpi} = -0.18$ (leeward controls)

(Figure 30.3-1)

Wind pressure (ASD) $p = q_z (G_{Cp} - G_{Cpi}) (0.6)$
 $= 86.0 (.87 - (-0.18)) (0.6)$
 $= 54.18 \text{ lbs/sq. ft.}$

Load at building column = $(54.18 \text{ lbs/sq.ft.})(210 \text{ sq. ft.})$
 $= 11.4 \text{ kips (horizontal)}$

SIDE RAIL AT STRUCTURAL PIPE BRACE

Max. wind area = 420 sq. ft.

Zone 4 Windward: $G_{Cp} = 0.72$ $G_{Cpi} = -0.18$
Zone 4 Leeward : $G_{Cp} = 0.82$ $G_{Cpi} = -0.18$ (leeward controls)

(Figure 30.3-1)

Wind pressure (ASD) $p = q_z (G_{Cp} - G_{Cpi}) (0.6)$
 $= 86.0 (.82 - (-0.18)) (0.6)$
 $= 51.6 \text{ lbs/sq. ft.}$

Load at building column = $(51.6 \text{ lbs/sq.ft.})(420 \text{ sq. ft.})$
 $= 21.7 \text{ kips (horizontal)}$