sCheck v3.2

A Steel Analysis and Design Program

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General Info

File Name	C:\CGInc\sCheck3\Examples-LRFD16\Example03-AiscDesignExamples-F5.sck
Design Code	AISC 360-22 (16th edition) LRFD
Using Direct Analysis Method	Yes
Consider Multiplier B1 for P-delta Effect	Yes
Date & Time	11/27/2023 20:36

Section Property - W12X58

Property	Value	Unit	Property	Value	Unit	Property	Value	Unit
A = Ag	17	in^2	bf	10	in	tf	0.64	in
tw	0.36	in	d	12.2	in	h / tw	27	
Cw	3570	in^6	h0	11.6	in	rts	2.81	in
Zx	86.4	in^3	Sx	78	in^3	Ix	475	in^4
rx	5.28	in	Zy	32.5	in^3	Sy	21.4	in^3
Iy	107	in^4	ry	2.51	in	J	2.1	in^4

Design Input

Input	Value	Unit	Input	Value	Unit	Input	Value	Unit
Pu = Pr	0	kips	Mux = Mxr	0	kip-ft	Muy = Myr	113	kip-ft
Cmx	1		Cmy	1		Vux	0	kips
Vuy	30	kips	Fy	50	ksi	Cb	1	
Lb	15	ft	Kx	1		Ky	1	
Kz	1		Lx	15	ft	Ly	15	ft
Lz	15	ft						

* Lcx = Kx * Lx; Lcy = Ky * Ly; Lcz = Kz * Lz

Axial Capacity Calculation

Step	Equation	Value	Note
Checking flange slenderness			
	b = bf / 2	5 in	
	b / tf	7.8125	
	$\lambda_r = 0.56\sqrt{\frac{E}{F_y}}$	13.487	
The section has non-slender flange element			
Checking web slenderness			
	b / t = h / tw	27	
	$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}$	35.884	
The section has non-slender web			
Compressive strength to account for flexural buckling			

	$\frac{K_x L_x}{r_x}$	34.091	
	$\frac{K_y L_y}{r_y}$	71.713	
	$\frac{KL}{r} = \max\left(\frac{K_x L_x}{r_x}, \frac{K_y L_y}{r_y}\right)$	71.713	
	$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$	55.654 ksi	Eq.E3-4
	$4.71\sqrt{\frac{E}{F_y}}$	113.43	
	$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$		
	$F_n = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$	34.329 ksi	Eq.E3-2
	$P_n = F_n A_g$	583.6 kips	Eq.E3-1
Compressive strength to account for torsional and flexural-torsional buckling			
	$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right) \frac{1}{I_x + I_y}$	94.6 ksi	Eq.E4-2
	$\frac{F_y}{F_e}$	0.52854	
	$\frac{F_y}{F_e} \leq 2.25$		
	$F_n = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$	40.077 ksi	Eq.E3-2
	$P_n = F_n A_g$	681.31 kips	Eq.E4-1
	Flexural buckling controls: Pn	583.6 kips	
	$\phi_c P_n$	525.24 kips	

Moment Magnification Calculation

Step	Equation	Value	Note
Stiffness reduction parameter			
	$\alpha = 1.00(\text{LRFD})$	1	
	Pr / Pns = Pr / (Fy * Ae)	0	
	$\alpha P_r / P_{ns} \leq 0.5$		
	$\tau_b = 1.0$	1	Eq.C2-2a
Moment magnifier B1 for P-delta effects in local x direction			

	$EI^* = 0.8\tau_b EI$	1.102e+07 ksi	
	$P_{e1} = \frac{\pi^2 EI^*}{(K_1 L)^2}$	3356.9 kips	Eq.A-8-5
	$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$	1	Eq.A-8-3
	Magnified Mux = Mux * B1	0 kip-ft	
Moment magnifier B1 for P-delta effects in local y direction			
	$EI^* = 0.8\tau_b EI$	2.4824e+06 ksi	
	$P_{e1} = \frac{\pi^2 EI^*}{(K_1 L)^2}$	756.18 kips	Eq.A-8-5
	$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$	1	Eq.A-8-3
	Magnified Muy = Muy * B1	113 kip-ft	
Mrx = Mux; Mry = Muy			

Major Flexure Capacity Calculation

Step	Equation	Value	Note
Web compactness:			
	$\lambda = \frac{h_c}{t_w}$	27	
	$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}}$	90.553	
	$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}}$	137.27	
Web is compact			
Flange compactness:			
	$\lambda = \frac{b_f}{2t_f}$	7.8125	
	$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$	9.1516	
	$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}}$	24.083	
Flange is compact			
Mnx to account for yielding			

	$M_n = M_p = F_y Z_x$	360 kip-ft	Eq.F2-1
Mnx to account for flange local buckling			
	$\lambda < \lambda_{pf}$		
	$M_n = M_p$	360 kip-ft	
Mnx to account for lateral-torsional buckling			
	$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$	8.8658 ft	Eq.F2-5
	For I section, c	1	
	$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$	29.849 ft	Eq.F2-6
	$M_n = M_p = F_y Z_x$	360 kip-ft	Eq.F2-1
Since Lp < Lb < Lr			
	$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$	321.26 kip-ft	Eq.F2-2
	Controlling nominal flexural strength Mnx	321.26 kip-ft	
	$M_{cx} = \phi_b M_{nx}$	289.14 kip-ft	

Minor Flexure Capacity Calculation

Step	Equation	Value	Note
Mny to account for yielding			
	Fy * Zy	135.42 kip-ft	
	Fy * Sy	89.167 kip-ft	
	$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y$	135.42 kip-ft	Eq.F6-1
Mny to account for lateral-torsional buckling			
	$\lambda < \lambda_{pf}$		
	$M_n = M_p$	135.42 kip-ft	
	Controlling nominal flexural strength Mny	135.42 kip-ft	
	$M_{cy} = \phi_b M_{ny}$	121.88 kip-ft	

Flexural and Axial Interaction Calculation

Step	Equation	Value	Note

	$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n}$	0	
	$\frac{P_r}{P_c} < 0.2$		
	$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$	0.92718	Eq.H1-1b
Axial-flexural strength: OK			

Major Shear Capacity Calculation

Step	Equation	Value	Note
	$A_w = dt_w$	4.392 in^2	
Computing Cv for major axis using G2.1			
	$k_v = 5.34$		
	h/t_w	27	
	$2.24\sqrt{E/F_y}$	53.946	
	$h/t_w \leq 2.24\sqrt{E/F_y}$		
	$C_{v1} = 1.0$		Eq.G2-2
Major shear strength			
	$V_n = 0.6F_y A_w C_{v1}$	131.76 kips	Eq.G2-1
	$h/t_w \leq 2.24\sqrt{E/F_y}$		
	$\phi_v = 1.00$		
	$\phi_v V_n$	131.76 kips	
	$\frac{V_u}{\phi_v V_n}$	0	
Shear strength (major axis): OK			

Minor Shear Capacity Calculation

Step	Equation	Value	Note
	$A_w = 2b_f t_f$	12.8 in^2	
Computing Cv2 for weak axis using G2.2			
	$k_v = 1.2$		
	$h/t_w = b/t_f$	7.8125	

$1.10\sqrt{k_v E/F_y}$	29.02	
$1.37\sqrt{k_v E/F_y}$	36.143	
$h/t_w \leq 1.10\sqrt{k_v E/F_y}$		
$C_{v2} = 1.0$	1	Eq.G2-9
Minor shear strength		
$V_n = 0.6F_y b_f t_f C_{v2}$	384 kips	Eq.G6-1
$\phi_v = 0.90$		
$\phi_v V_n$	345.6 kips	
$\frac{V_u}{\phi_v V_n}$	0.086806	
Shear strength (minor axis): OK		