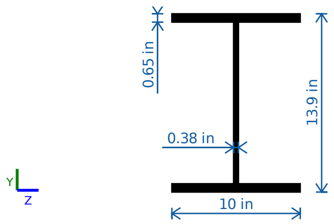
Input Summary

Input	Description	Value
DM	**Design Method**	LRFD
d	**Depth (D)** *Depth of Member*	13.9 in
bt	**Flange Width (B_t)**	10 in
tt	**Flange Thickness (t_t)**	0.65 in
tw	**Web Thickness (t_w)** *Web Thickness*	0.38 in
Fy	**F_y** *Yield Strength of Steel*	50 ksi
Fu	**F_u** *Ultimate Strength of Steel*	65 ksi
kz	**k_z** *Effective length factor, major axis*	1
lz	**L_z** *Major Axis*	30 ft
ky	**k_y** *Effective length factor, minor axis*	1
ly	**L_y** *Minor Axis*	15 ft
lb	**L_b** *Beam Unsupported Lateral Length, Lb*	1 ft
L_t	**L_t** *Unsupported length, torsional buckling*	30 ft
Cb	**C_b** *Cb for UDL w/o Lateral Bracing Along Span*	1.14
Put	**P_{ut}** *Axial Tension*	100 kip
Puc	**P_{uc}** *Axial Compression*	120 kip
Muz	**M_{uz}** *Moment about Major Axis*	70 kip-ft
Muy	**M_{uy}** *Moment about Minor Axis*	70 kip-ft
Vuz	**V_{uz}** *Shear Force about Major Axis*	0 kip
Vuy	**V_{uy}** *Shear Force about Minor Axis*	0 kip

Section Dimensions



Section Properties

Gross Area	$A =$	$17.79 in^2$
Moment of Inertia about Major Axis	$I_z =$	$634.38 in^4$
Moment of inertia about Minor Axis	$I_y =$	$108.39 in^4$
Elastic Section Modulus about Major Axis	$S_z =$	$101.21 in^3$
Elastic section Modulus about Minor Axis	$S_y =$	$32.95 in^3$
Radius of Gyration about Major Axis	$r_z =$	5.97in
Radius of Gyration about Minor Axis	$r_y =$	2.47in
Plastic Section Modulus about Major Axis	$Z_z =$	$101.21 in^3$
Plastic Section Modulus about Minor Axis	$Z_y =$	$32.95 in^3$
Plastic Section Modulus about Minor Axis	$J =$	$2.06 in^4$

Warping Constant $C_w =$ 4757.3in^6

Tensile Capacity

Section D1 - Slenderness limitations

D1, AISC 360-16

$$max\left(\frac{k_z * L_z}{r_z}, \frac{k_y * L_y}{r_y}\right) \leq 300$$

$$max\left(\frac{360.00}{5.97}, \frac{180.00}{2.47}\right) \leq 300$$

$$max(5.03, 6.07) \leq 300$$

Slenderness is acceptable

Section D2 - Tensile Strength

D2-1, AISC 360-16

$$A_g = 2 * (b_t * t_t) + tw * (d - 2 * t_t)$$

$$A_g = 2 * (10 * 0.65) + 0.38 * (13.9 - 2 * 0.65)$$

$$A_g = 17.8in^2$$

$$P_t = A_g * Fy = 17.8 * 50 = 889.4kip$$

Design Tensile Strength

$$\Phi_t P_t = \Phi_t * P_t = 0.9 * 889.400 = 800.5kip$$

Utilization Ratio

$$UR_{Pt} = \frac{P_{ut}}{\Phi_t P_t} = \frac{100}{800.460} = 0.125$$

UTILITY: 0.12

Compression Capacity

Chapter B - Classification of Section:

$$\lambda = b/t = \frac{(\frac{b_t}{2})}{t_t} = \frac{(\frac{10}{2})}{0.65} = 7.692$$

$$\lambda_r = 0.56 * \sqrt{\frac{E}{F_y}} = 0.38 * \sqrt{\frac{29000}{50}} = 13.487$$

$$\lambda \leq \lambda_r \rightarrow \text{Flange is non-slender}$$

$$\lambda = h/t_w = \frac{h}{t_w} = \frac{12.6}{0.38} = 33.158$$

$$\lambda_r = 1.49 * \sqrt{\frac{E}{F_y}}r = 1 * \sqrt{\frac{29000}{50}} = 35.884$$

$$\lambda \leq \lambda_r \rightarrow \text{Web is non-slender}$$

Section E3 - Flexural buckling of members

Cl.E3, AISC 360-16

$$F_{ez} = \frac{(\pi)^2 \times E}{\left(\frac{k_z \times L_z}{r_z}\right)^2} = \frac{(3.14)^2 \times 29000\ ksi}{\left(\frac{1 \times 360\ in}{6\ in}\right)^2} = 78.712\ ksi$$

Cl.E3, AISC 360-16

$$F_{ey} = \frac{(\pi)^2 \times E}{\left(\frac{k_y \times L_y}{r_y}\right)^2} = \frac{(3.14)^2 \times 29000\ ksi}{\left(\frac{1 \times 180\ in}{2.5\ in}\right)^2} = 53.895\ ksi$$

Section E4 - Torsional buckling of members

Cl.E4, AISC 360-16

$$F_{et} = \frac{\left(\frac{(\pi)^2 \times E \times C_w}{(L_t)^2} + G \times J\right)}{(I_z + I_y)} = \frac{\left(\frac{(3.14)^2 \times 29000\,ksi \times 4757.3\,in^6}{(360\,in)^2} + 11200\,ksi \times 2.06\,ksi\right)}{(634.38\,in^4 + 108.39\,in^4)}$$

$$F_{et} = 45.207\,ksi$$

Elastic Critical Buckling Stress

Eq.E3-4, AISC 360-16

$$F_e = min(F_{ez}, F_{ey}, F_{et}) = 45.207 ksi$$

Eq.E3-2, AISC 360-16

$$F_{cr} = \left(0.658^{\left(\frac{F_y}{F_e}\right)}\right) * F_y$$

$$F_{cr} = \left(0.658^{\left(\frac{50}{45.207}\right)}\right) * 50$$

$$F_{cr} = 31.472 ksi$$

Nominal Compressive Strength

Eq.E3-1, AISC 360-16

$$P_n = F_{cr} * A$$

$$P_n = 31.472 * 17.79$$

$$P_n = 559.886 kips$$

Design Compressive Strength

Cl.E1, AISC 360-16

$$\Phi_C P_n = \Phi_c * P_n$$

$$\Phi_C P_n = 0.9 * 559.886$$

$$\Phi_C P_n = 503.898 kip$$

Utilization Ratio

$$UR_{Pn} = \frac{P_{uc}}{\Phi_C P_n} = \frac{120}{503.898} = 0.238$$

UTILITY:
0.24

Moment Capacity about Major Axis

Chapter B - Classification of Section:

$$\lambda = b/t = \frac{(\frac{b_t}{2})}{t_t} = \frac{(\frac{10}{2})}{0.65} = 7.692$$

$$\lambda_p = 0.38 * \sqrt{\frac{E}{F_y}} = 0.38 * \sqrt{\frac{29000}{50}} = 9.152$$

$$\lambda_r = 1 * \sqrt{\frac{E}{F_y}} r = 1 * \sqrt{\frac{29000}{50}} = 24.083$$

$$\lambda \leq \lambda_p \rightarrow \text{Flange is Compact}$$

$$\lambda = h/t_w = \frac{h}{t_w} = \frac{12.60}{0.38} = 33.158$$

$$\lambda_p = 3.76 * \sqrt{\frac{E}{F_y}} = 3.76 * \sqrt{\frac{29000}{50}} = 90.553$$

$$\lambda_r = 5.7 * \sqrt{\frac{E}{F_y}} = 5.7 * \sqrt{\frac{29000}{50}} = 137.274$$

$$\lambda \leq \lambda_p \rightarrow \text{Web is Compact}$$

Section F2 - Doubly symmetric compact I-shaped members

1. Yielding

Eq.F2-1, AISC 360-16

$$M_{nz1} = M_p = \frac{F_y * Z_z}{12} = 50 * \frac{101.21}{12} = 421.708 kip - ft$$

2. Lateral-Torsional Buckling

$$L_b = 1 ft$$

Eq.F2-5, AISC 360-16

$$L_p = \frac{1.76 * r_y * \sqrt{\frac{E}{F_y}}}{12} = \frac{1.76 * 2.470 * \sqrt{\frac{29000}{50}}}{12} = 8.725 ft$$

$$r_{ts}^2 = \frac{\sqrt{I_y * C_w}}{S_z} = \frac{\sqrt{108.39 * 4757.3}}{101.21} = 7.095 in^2$$

$$r_{ts} = \sqrt{r_{ts}^2} = \sqrt{7.095} = 2.664 in$$

Eq.F2-6, AISC 360-16

$$L_{r1} = 1.95 * r_{ts} * \frac{E}{0.7 * F_y}$$

$$L_{r1} = 1.95 * 2.664 * \frac{29000}{50}$$

$$L_{r1} = 358.640$$

Eq.F2-6, AISC 360-16

$$L_r = L_{r1} * \sqrt{\frac{J_c}{S_z * h_0} + \sqrt{\left(\frac{J_c}{S_z * h_0}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$$L_r = L_{r1} * \sqrt{\frac{2.06 * 1}{101.2 * 13.3} + \sqrt{\left(\frac{2.06 * 1}{101.2 * 13.3}\right)^2 + 6.76\left(\frac{0.7 * 50}{29000}\right)^2}}$$

$$L_r = 25.435 ft$$

Eq.F2-1, AISC 360-16

$$M_{nz2} = M_p = 421.708 kip - ft$$

Moment Capacity summary

Eq.F2-2, AISC 360-16

$$M_{nz} = min(M_{nz1}, M_{nz2})$$

$$M_{nz} = min(421.708, 421.708)$$

$$M_{nz} = 421.708 kip - ft$$

Design Moment Capacity (LRFD)

$$\Phi_b M_{nz} = \Phi_b * Mnz$$

$$\Phi_b M_{nz} = 0.9 * 421.708$$

$$\Phi_b M_{nz} = 379.537 kip - ft$$

Utilization Ratio

$$UR_{Mnz} = \frac{M_{uz}}{\Phi_b M_{nz}} = \frac{70}{379.537} = 0.184$$

UTILITY: 0.18

Moment Capacity about Minor Axis

1. Yielding

Eq.F6-1, AISC 360-16

$$M_{ny1} = M_{py} = F_y * Z_y = 50 * 32.950 = 137.292 Kip - ft$$

2. Flange local buckling

Eq.F6-2, AISC 360-16

$$M_{ny2} = M_{ny1}$$
$$M_{ny2} = 137.292 Kip - ft$$

Moment capacity summary

$$M_{ny} = min(M_{ny1}, M_{ny2})$$
$$M_{ny} = min(137.292, 137.292)$$
$$M_{ny} = 137.292 kip - ft$$

Design Moment Capacity

$$\Phi_b M_{ny} = \Phi_b * M_{ny}$$
$$\Phi_b M_{ny} = 0.9 * 137.292$$
$$\Phi_b M_{ny} = 123.563 kip - ft$$

Utilization Ratio

$$UR_{Mny} = \frac{M_{uy}}{\Phi_b M_{ny}} = \frac{70}{123.563} = 0.567$$

UTILITY: 0.57

Shear Capacity about Major Axis

$$A_w = d * t_w$$
$$A_w = 13.9 * 0.38$$
$$A_w = 5.282 in^2$$

$$\lambda = 33.158$$

Limit 1

$$l_1 = 2.24 * \sqrt{\frac{E}{F_y}} = 2.24 * \sqrt{\frac{29000}{50}} = 53.946$$

Limit 2

$$l_2 = 1.1 * \sqrt{\frac{5.34 * E}{F_y}} = 1.1 * \sqrt{\frac{5.3429000}{50}} = 61.218$$

Nominal Shear Capacity

Eq.G2-1, AISC 360-16

$$V_{nz} = 0.6 * F_y * A_w * C_{v1}$$
$$V_{nz} = 0.6 * 50 * 5.282 * 1.000$$
$$V_{nz} = 158.460 kip$$

Design Shear Capacity

$$\Phi_v V_{nz} = \Phi_v * V_{nz}$$
$$\Phi_v V_{nz} = 1 * 158.460$$
$$\Phi_v V_{nz} = 158.460 kip$$

Utilization Ratio

$$UR_{Vnz} = \frac{V_{uz}}{\Phi_v V_{nz}} = \frac{0}{158.46} = 0.000$$

Shear Capacity about Minor Axis

$$A_f = 2 * b_t * t_t$$
$$A_f = 2 * 10 * 0.65$$
$$A_f = 13.000 in^2$$

G2.2, AISC 360-16

Limit 2

$$l_2 = 1.1 * \sqrt{\frac{k_v * E}{F_y}} = 1.1 * \sqrt{\frac{1.2 * 29000}{50}} = 29.020$$

Nominal Shear Capacity

G6-1, AISC 360-16

$$V_{ny} = 0.6 * F_y * A_f * C_{v2}$$

$$V_{ny} = 0.6 * 50 * 13.000 * 1.000$$

$$V_{ny} = 390.000 kip$$

Design Shear Capacity

$$\Phi_v V_{ny} = \Phi_v * V_{ny}$$

$$\Phi_v V_{ny} = 0.9 * 390.000$$

$$\Phi_v V_{ny} = 351.000 kip$$

Utilization Ratio

$$UR_{Vny} = \frac{V_{uy}}{\Phi_v V_{ny}} = \frac{0}{351} = 0.000$$

H1. Flexure and compression interaction

H1-1a, AISC 360-16

$$UCM = \frac{P_r}{P_c} + \frac{8}{9} \times \left(\frac{M_{rz}}{M_{cz}} + \frac{M_{ry}}{M_{cy}}\right)$$

$$UCM = \frac{120\ kip}{503.898\ kip} + \frac{8}{9} \times \left(\frac{70\ kip - ft}{379.537\ kip - ft} + \frac{70\ kip - ft}{123.563\ kip - ft}\right)$$

$$UCM = 0.91$$

UTILITY: 0.91

H1. Flexure and tension interaction

H1-1a, AISC 360-16

$$UTM = \frac{T_r}{(2 \times T_c)} + \frac{M_{rz}}{M_{cz}} + \frac{M_{ry}}{M_{cy}}$$

$$UTM = \frac{100\ kip}{(2 \times 800.46\ kip)} + \frac{70\ kip - ft}{379.537\ kip - ft} + \frac{70\ kip - ft}{123.563\ kip - ft}$$

$$UTM = 0.81$$

UTILITY: 0.81

Results Summary

Result Name	**Results**
UTILITY RATIOS	
Tension Utility (UR_{Pt})	0.12
Compression Utility (UR_{Pn})	0.24
Major Bending Utility (UR_{Mnz})	0.18
Minor Bending Utility (UR_{Mny})	0.57
Major Shear Utility (UR_{Vnz})	0.00
Minor Shear Utility (UR_{Vny})	0.00
Compression/Flexion Interaction (UCM)	0.91
Tension/Flexion Interaction (UTM)	0.81
CAPACITIES	
Tension Section Capacity	800.46 kip
Compression Section Capacity	503.90 kip
Major Bending Capacity	379.54 kip-ft
Minor Bending Capacity	123.56 kip-ft
Major Shear Capacity	351.00 kip
Minor Shear Capacity	158.46 kip

About this Calculator

Calculator Name: AISC 360-16 Steel I-Beam Design
Description: Calculate the capacities of an I section beam based on its depth, length, and width. The capacities calculated include maximum moment capacity, maximum shear capacity, and maximum deflection capacity. This tool uses the American Institute of Steel Construction (AISC) codes and standards for the calculations.

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