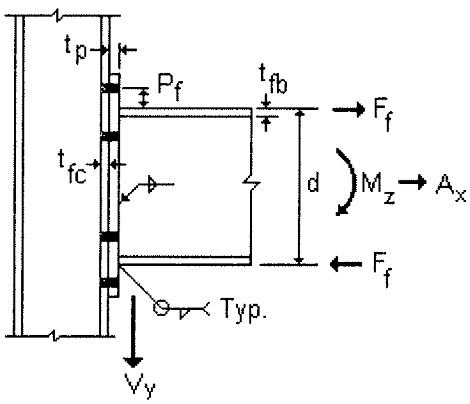
MOMENT CONNECTION DESIGN PER ASD- 9th ED.
UNFACTORED LOAD
(AMERICAN CODE)

Moment Connection - Four Tension Bolt UnStiffened End-Plate-MC (ASD - 9th Edition)

1.0 BEAM, COLUMN, END PLATE, WELD AND BOLT PROPERTIES:



1.1 Beam Properties Canadian Section, W310 x 107:

Yield strength of beam material, $F_{yb} = 350MPa$

Tensile strength of beam material, $F_{ub} = 450MPa$

Depth, $d_{bm} = 311mm$ Web thickness, $t_{wb} = 10.9mm$

Root distance from center of web, $k_{1b} = 26mm$

Width, $b_{fb} = 306.mm$ Flange thickness, $t_{fb} = 17mm$

Root distance from top of flange, $k_b = 39 \times mm$

Beam gage, $gage_b = 130mm$ Clear height between flanges, $T_b = 233mm$

1.2 Column Properties Canadian Section, W310 x 107 :

Yield strength of column material, $F_{yc} = 350MPa$ Tensile strength of column material, $F_{uc} = 450MPa$

Depth, $d_c = 311mm$ Web thickness, $t_{wc} = 10.9mm$ Root distance from center of web, $k_{1c} = 26mm$

Width, $b_{fc} = 306mm$ Flange thickness, $t_{fc} = 17mm$ Root distance from top of flange, $k_c = 39mm$

Beam gage, $gage_c = 130mm$ Clear height between flanges, $T_c = 233mm$

1.3 End plate Properties:

Yield strength of plate material, $F_{yp} = 300MPa$ Tensile strength of plate material, $F_{up} = 450MPa$

1.4 Bolt Properties: (Refer Table J3.2 of ASD-9th edition, pg 5-73)

Bolt material A325M (per AISC-ASD Pg 4-122)

Bolt diameter: $BoltDia = 25mm$ Bolt gage: $gage = 130mm$

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Bolt center to flange distance, Pf ($P_f \le 60mm$): $P_f = 50mm$

Allowable shear per bolt, $F_v = 130MPa$ Allowable tension per bolt, $F_t = 300MPa$

Minimum edge distance required for end plate, (Section J3-9 of ASD, pg 5-75) $ED_{pl} = 1.5 \times BoltDia$ $ED_{pl} = 38\,mm$

Minimum edge distance required -assuming sheared edges, (Section J3-9 of ASD, pg 5-75) $ED_{col} = 1.5 \times BoltDia$ $ED_{col} = 38\,mm$

1.5 Welding Electrodes : E70XX

Allowable stress on welds, $F_{y\_weld} = 0.3 \times 480MPa$ $F_{y\_weld} = 144\,MPa$

2.0 LOAD DATA (Un-factored Load): (From different load combinations)

Axial , $A_x = 50kN$ Major axis moment, $M_z = 100kN \times m$

Vertical shear, $V_y = 50kN$ Horz. shear, $V_z = 20kN$

Allowable stress increase factor (1.33 for wind or seismic combinations): $\alpha = 1.00$

3.0 MOMENT CONNECTION DESIGN:

3.1 Bolt Design :

Flange force: $FF = \dfrac{M_z}{d_{bm} - t_{fb}} + \dfrac{A_x}{2}$ $FF = 365.1\,kN$

Allowable forces at tension flange (using equation K1-1): $Pallow_{bf\_3} = \dfrac{t_{fc}^{\,2} \times F_{yc}}{0.16}$ $Pallow_{bf\_3} = 632\,kN$

Check Column Size: Column_size = "REVISE" if $Pallow_{bf\_3} < FF$; "OK" otherwise Column_size = "OK"

Tension per bolt : $T_{bolt} = \dfrac{FF}{4}$ $T_{bolt} = 91.3\,kN$

Shear per bolt : $V_{bolt} = \dfrac{\sqrt{V_y^{\,2} + V_z^{\,2}}}{8}$ $V_{bolt} = 6.7\,kN$

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Actual shear stress per bolt, $f_v = \dfrac{V_{bolt}}{0.25 \times \pi \times BoltDia^2}$ $f_v = 13.71\ \text{MPa}$

Actual tensile stress per bolt, $f_t = \dfrac{T_{bolt}}{0.25 \times \pi \times BoltDia^2}$ $f_t = 185.96\ \text{MPa}$

<u>With reference to Table J 3.3 ref. ASD</u>

Allowable tensile stress , $F_{t\_all} = \sqrt{(F_t \times \alpha)^2 - 4.39 \times f_v^2}$ $F_{t\_all} = 298.62\ \text{MPa}$

Bolt interaction for shear force, $Int_{bolt1} = \dfrac{f_v}{F_v}$ $Int_{bolt1} = 0.11$

Bolt interaction for tension force, $Int_{bolt2} = \dfrac{f_t}{F_{t\_all}}$ $Int_{bolt2} = 0.623$

Bolt_Dia = | "REVISE BOLT DIA" if $Int_{bolt1} > 1.0$
| "REVISE BOLT DIA" if $Int_{bolt2} > 1.0$
| "SAFE" otherwise

Bolt_Dia = "SAFE"

Check column flange bearing: Col_FlngBrg = | "Use Thicker Column" if $\dfrac{1.2 \times F_{uc} \times BoltDia \times t_{fc}}{V_{bolt}} < 1.0$
| "Column Flange OK" otherwise

Col_FlngBrg = "Column Flange OK"

3.2 End Plate Design :

Flange weld: (Refer AISC-ASD, page 4-100 for E70XX Electrode)

$$D_f = \frac{FF}{0.707 \times F_{y\_weld} \times \alpha \times (2 \times b_{fb} - t_{wb})} \qquad D_f = 6\,mm$$

Maximum effective width of plate to be used in design : (Ref : AISC-ASD, Pg. 4-117)

Plate width: $A_p = ceil\left(\frac{b_{fb} + 25 \times mm}{10 \times mm}\right) \times 10 \times mm$ $\qquad A_p = 340\,mm$

Distance from the bolt center to the face of the flange $\qquad P_f = 50\,mm$

Effective bolt distance: $P_e = P_f - \left(\frac{BoltDia}{4}\right) - (0.707 \times D_f)$ $\qquad P_e = 39.53\,mm$

C_a depends on grade: $C_a = 1.13$ $\qquad$ C_b factor: $C_b = \sqrt{\frac{b_{fb}}{A_p}}$ $\qquad C_b = 0.95$

Area of tension flange: $A_f = b_{fb} \times t_{fb}$ $\qquad A_f = 5202\,mm^2$

Web area, w/o flanges: $A_w = [d_{bm} - (2 \times t_{fb})] \times t_{wb}$ $\qquad A_w = 3019.3\,mm^2$

Factor α_m: $\alpha_m = C_a \times C_b \times \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \times \left(\frac{P_e}{BoltDia}\right)^{\frac{1}{4}}$ $\qquad \alpha_m = 1.44$

Moment Resisted, $M_e = \alpha_m \times P_e \times \frac{FF}{4}$ $\qquad M_e = 5.2\,kN \times m$

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Determine End- Plate Thickness:

Tee- Stub Analogy Thickness: $t_{pa} = \sqrt{\frac{6 \times M_e}{(0.75 \times F_{yp}) \times A_p \times \alpha}}$ $t_{pa} = 20.2\,mm$

Plate thickness from bearing: $t_{pb} = \frac{V_{bolt}}{1.2 \times F_{up} \times BoltDia}$ $t_{pb} = 0.5\,mm$

End-Plate Thickness: $t_p = ceil\left[\frac{max((t_{pa}\ \ t_{pb}))}{5mm}\right] \times 5mm$ $t_p = 25\,mm$

Calculation of the minimum weld size in accordance with the AISC Table J2.4 based on a beam flange thickness of $t_{fb} = 17\,mm$ and the end plate thickness of $t_p = 25\,mm$.

$Thick = max\left(\begin{pmatrix} t_p \\ t_{fb} \end{pmatrix}\right)$ $Thick = 25\,mm$ $Thin = min\left(\begin{pmatrix} t_p \\ t_{fb} \end{pmatrix}\right)$ $Thin = 17\,mm$

$Wf_{min} = if(Thick > 19mm, 8mm, if(Thick > 13mm, 6mm, if(Thick > 6mm, 5mm, 3mm)))$ $Wf_{min} = 8\,mm$

Recheck weld size D_{f1} based on end-plate thickness: $D_{f1} = max((Wf_{min}\ \ D_f))$ $D_{f1} = 8\,mm$ fillet weld

Plate depth: $B_p = ceil\left(\frac{d_{bm} + 2 \times P_f + 2 \times ED_{pl}}{10 \times mm}\right) \times 10 \times mm$ $B_p = 490\,mm$

Beam Web To End Plate Weld :

Required weld to develop maximum web tension stress (0.60 Fy) in web near flanges:

$D_{wt} = \frac{0.60 \times F_{yb} \times t_{wb}}{2 \times (0.707 F_{y\_weld} \times \alpha)}$ $D_{wt} = 11.24\,mm$

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Required weld to resist applied shear force [Ref. AISC Vol. II, Connections pg. 4-29]:

$$D_{ws} = \frac{\sqrt{V_y^2 + V_z^2}}{2 \times \left(0.707F_{y\_weld} \times \alpha\right) \times \min\left[\left[\left(\frac{d_{bm}}{2} - t_{fb}\right) \quad \left(d_{bm} - t_{fb} - P_f - 2 \times \text{BoltDia}\right)\right]\right]} \qquad D_{ws} = 2\,\text{mm}$$

Calculation of the minimum weld size in accordance with the AISC Table J2.4 based on a beam web thickness of $t_{wb} = 10.9\,\text{mm}$ and the end plate thickness of $t_p = 25\,\text{mm}$.

$$\text{Thick}_1 = \max\left(\begin{pmatrix} t_p \\ t_{wb} \end{pmatrix}\right) \quad \text{Thick}_1 = 25\,\text{mm} \qquad \text{Thin}_1 = \min\left(\begin{pmatrix} t_p \\ t_{wb} \end{pmatrix}\right) \quad \text{Thin}_1 = 10.9\,\text{mm}$$

$$Ww_{min} = \text{if}(\text{Thick} > 19\text{mm}, 8\text{mm}, \text{if}(\text{Thick} > 13\text{mm}, 6\text{mm}, \text{if}(\text{Thick} > 6\text{mm}, 5\text{mm}, 3\text{mm}))) \qquad Ww_{min} = 8\,\text{mm}$$

Use the following size fillet weld on both sides of beam web

$$D_{w1} = \max\left(\left(\min\left(D_{f1}, D_{wt}\right) \quad D_{ws} \quad Ww_{min}\right)\right) \qquad D_{w1} = 8\,\text{mm}$$

Note : The web weld shall not exceed flange weld if it comes from the maximum web tension criteria :

3.3 Column Stiffeners requirement:

3.3.1 Check column web yielding opposite the compression flange :

$$P_{bf} = \left(\frac{5}{3}\right) \times FF \qquad P_{bf} = 608.56\,\text{kN}$$ (Ref : AISC-ASD, K1-1)

$$\text{Pallow}_{bf\_1} = F_{yc} \times t_{wc} \times \left[t_{fb} + \left(6 \times k_c\right) + 2 \times t_p + 2 \times 1\right. \quad \text{Pallow}_{bf\_1} = 1209.4\,\text{kN}$$ (Refer AISC Design Guides, Series-4)

$$\text{Ratio1} = \frac{P_{bf}}{\alpha \times \text{Pallow}_{bf\_1}} \qquad \text{Ratio1} = 0.5$$

If Ratio1 is > 1, column requires stiffeners oppposite both the tension and compression flange connection of the beam.

3.3.2 Check column web buckling:

$$Pallow_{bf\_2} = \frac{4100 \times t_{wc}^{3} \times \sqrt{F_{yc}}}{T_c} \times \left(\frac{\sqrt{kip}}{in}\right) \qquad Pallow_{bf\_2} = 1119.4\,kN$$

$$Ratio2 = \frac{P_{bf}}{\alpha \times Pallow_{bf\_2}} \qquad Ratio2 = 0.544$$

If Ratio2 is > 1, column requires stiffeners opposite the compression flange.

3.3.3 Check Column Flange Bending Opposite Tension Flange:

$$b_{pc} = (t_{fb} + 2 \times P_f) \times 2.5 \qquad b_{pc} = 292.5\,mm$$

$$P_{ec} = \left(\frac{gage}{2}\right) - k_{1b} - \left(\frac{BoltDia}{4}\right) \qquad P_{ec} = 32.75\,mm$$

C_a depends on beam grade: $C_a = 1.13$

$$\alpha_{m1} = C_a \times (1) \times 1^{\frac{1}{3}} \times \left(\frac{P_{ec}}{BoltDia}\right)^{0.25} \qquad \alpha_{m1} = 1.21$$

$$M_e = \alpha_{m1} \times FF \times \frac{P_{ec}}{4} \qquad M_e = 3.6\,kN \times m$$

$$t_{pc} = \sqrt{\frac{6 \times M_e}{0.75 \times F_{yc} \times b_{pc} \times \alpha}} \qquad t_{pc} = 16.81\,mm$$

(Required flange thickness)

$$Ratio3 = \frac{t_{pc}}{t_{fc}} \qquad Ratio3 = 0.99$$

If Ratio3 is > 1, column flange requires stiffeners opposite the tension flange.

Check Column Stiffener requirement:

Column_stiffener =	"REVISE COLUMN OR BEAM" if Ratio1 > 1.0
	"REVISE COLUMN OR BEAM" if Ratio2 > 1.0
	"REVISE COLUMN OR BEAM" if Ratio3 > 1.0
	"NOT REQUIRED" otherwise

Column_stiffener = "NOT REQUIRED"

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3.4 **Column Web Shear Yielding and requirement of web doubler plate:** (Ref : AISC-ASD, Pg. 4-107)

Required col. web thickness: $t_{req} = \dfrac{M_z}{0.95 \times d_{bm} \times d_c \times (0.4 \times \alpha \times F_{yc})}$ $t_{req} = 7.77\ \text{mm}$

Check Column web Stiffener requirement:

Web_stiffener =	"REVISE COLUMN OR BEAM" if $t_{req} > t_{wc}$
	"NOT REQUIRED" otherwise

Web_stiffener = "NOT REQUIRED"

4.0 **Summary of End Plate Design:**

End Plate thickness: $t_p = 25\ \text{mm}$ End Plate width: $A_p = 340\ \text{mm}$ End Plate depth: $B_p = 490\ \text{mm}$

Web weld: $D_{w1} = 8\ \text{mm}$ Flange weld: $D_{f1} = 8\ \text{mm}$ ***(if weld size>12mm, use Butt weld)***

Distance from centerline of bolt to edge of end plate: $ED_{pl} = 37.5\ \text{mm}$ $ED_{col} = 37.5\ \text{mm}$

Distance from centerline of bolt to nearest surface of tension flange: $P_f = 50\ \text{mm}$

Bolt gage: $gage = 130\ \text{mm}$ Bolt diameter: $BoltDia = 25\ \text{mm}$