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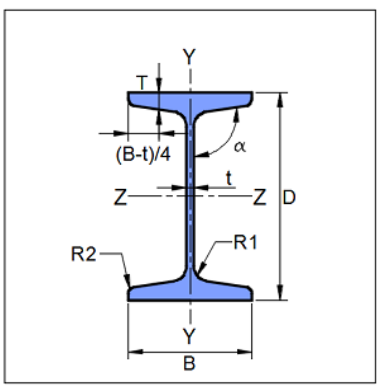
Company Name	IIT Bombay	Project Title	Sample Connection Design
Group/Team Name	Osdag	Subtitle	Base Plate Connection
Designer	Engineer #1	Job Number	1.3.2.2
Date	18 /12 /2020	Client	Pradyumna M, Independent Consultant, Bengaluru

1 Input Parameters

Main Module	Moment Connection
Module	Base Plate
Connectivity	Moment Base Plate
End Condition	Fixed
Axial Compression (kN)	1600.0
Axial Tension/Uplift (kN)	0.0
Shear Force (kN)	
- Along major axis (z-z)	80.0
- Along minor axis (y-y)	23.0
Bending Moment (kNm)	
- Major axis (M_{z-z})	180.0
- Minor axis (M_{y-y})	45.0

Column Section - Mechanical Properties



Column Section		HB 450	
Material		E 300 (Fe 440)	
Ultimate Strength, f_u (MPa)		440.0	
Yield Strength, f_y (MPa)		300.0	
Mass, m (kg/m)	87.22	I_z (cm^4)	39200.0
Area, A (cm^2)	111.0	I_y(cm^4)	2980.0
None	None	r_z (cm)	18.7
D (mm)	450.0	r_y (cm)	5.18
B (mm)	250.0	Z_z (cm^3)	1740.0
T (mm)	13.7	Z_y (cm^3)	238.0
t (mm)	9.8	Z_{pz} (cm^3)	1950.0
Flange Slope	94	Z_{py} (cm^3)	394.0
R_1 (mm)	15.0		
R_2 (mm)	7.5		

Base Plate - Design Preference

Material	E 300 (Fe 440)
Ultimate Strength, f_u (MPa)	440
Yield Strength, f_y (MPa)	280

Stiffener/Shear Key - Design Preference	
Material	E 300 (Fe 440)
Ultimate Strength, f_u (MPa)	440
Yield Strength, f_y (MPa)	300
Anchor Bolt Outside Column Flange - Input and Design Preference	
Diameter (mm)	['M30']
Property Class	['10.9']
Anchor Bolt Type	End Plate Type
Anchor Bolt Galvanized?	Yes
Designation	M30X532.7 IS5624 GALV
Hole Type	Over-sized
Total Length (mm)	525.7
Material Grade, f_u (MPa)	1040.0
Anchor Bolt Inside Column Flange - Input and Design Preference	
Diameter (mm)	N/A
Property Class	N/A
Anchor Bolt Type	N/A
Anchor Bolt Galvanized?	N/A
Designation	N/A
Hole Type	N/A
Total Length (mm)	N/A
Material Grade, f_u (MPa)	N/A
Friction Coefficient (between concrete and anchor bolt)	0.3
Weld - Design Preference	
Type of Weld Fabrication	Shop Weld
Material Grade Overwrite, f_u (MPa)	600.0
Detailing - Design Preference	
Edge Preparation Method	b - Rolled, machine-flame cut, sawn and planed
Are the Members Exposed to Corrosive Influences?	No
Design - Design Preference	
Design Method	Limit State Design
Base Plate Analysis	Elastic Analysis Method

2 Design Checks

Design Status	Pass

2.1 Design Parameters

Check	Required	Provided	Remarks
Bearing Strength of Concrete (N/mm^2)		$\sigma_{br} = 0.45fck$ $= 0.45 \times 25$ $= 11.25$ $[Ref.\ IS\ 456:2000,\ Cl.\ 34.4]$	**OK**
Grout Thickness (mm)		t_g = 50	**OK**
Modular Ratio		$E_s = 2 \times 10^5\ (N/mm^2)$ $E_c = 5000\ \sqrt{f_{ck}}\ (N/mm^2)$ $= 5000\ \times \sqrt{25}\ =\ 25000.0$ $n = \frac{E_s}{E_c}$ $n = \frac{200000}{25000.0}$ $= 8.0$ $[Ref.\ IS\ 800:2007,\ IS\ 456:2000]$	**OK**
Epsilon - stiffener plate		$\epsilon_{st} = \sqrt{\frac{250}{f_{y\,st}}}$ $= \sqrt{\frac{250}{300}}$ $= 0.91$ $[Ref.\ IS\ 800:2007,\ Table\ 2]$	**OK**

2.2 Load Consideration

Check	Required	Provided	Remarks
Axial Compression (kN)	$P_x = 1600.0$	$P_u = max(P_x,\ 0.3P_d),\ but,\ \leq P_d$ $= max(1600.0,\ 0.3 \times 3027.27)$ $= max(1600.0,\ 908.18)$ ≤ 3027.27 $= 1600.0$ $[Ref. IS\ 800:2007, Cl.\ 10.7]$ $Note:\ P_d\ is\ the\ design\ axial\ capacity\ of\ the\ column$	Pass
Axial Tension/Uplift (kN)		$P_{up} = 0.0$	OK
Shear Force - along major (z-z) axis (kN)	$V_d = 391.27$	$V_1 = 80.0$	Pass
Shear Force - along minor (y-y) axis (kN)	$V_d = 391.27$	$V_2 = 23.0$	Pass
Bending Moment - major (z-z) axis (kNm)	$M_z = 180.0$	$M_{zmin} = 0.5 * M_{dz-z}$ $= 0.5 \times 474.55$ $= 237.27$ $M_{uzz} = max(M_z,\ M_{zmin}),\ but, \leq M_{dz-z}$ $= max(180.0,\ 237.27)$ ≤ 474.55 $= 223.66$ $Note: The\ column\ is\ classified\ as\ semi-compact$ $[Ref.\ IS\ 800:2007,\ Cl.\ 8.2.1.2]$	Pass
Bending Moment - minor (y-y) axis (kNm)	$M = 45.0$	$M_{yy} = 45.0$	Pass

Check	Required	Provided	Remarks
Interaction Ratio	IR < 1.0	$IR,\ axial = P_x/P_d$ $= 1600.0/3027.27$ $= 0.529$ $IR,\ moment = M_z/M_{d_{zz}}$ $= 180.0/474.55$ $= 0.379$ $IR,\ sum = IR,\ axial + IR,\ moment$ $= 0.529 + 0.379$ $= 0.908$	Pass

2.3 Plate Washer and Nut Details - Anchor Bolt Outside Column Flange

Check	Required	Provided	Remarks
Plate Washer Size (mm)		$Square - 58X58$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	Pass
Plate Washer Thickness (mm)		$t_w = 8.5$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	Pass
Plate Washer Hole Diameter (mm)		$d_h = 33$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	Pass
Nut (hexagon) Thickness (mm)		$t_n = 25.6$ $[Ref.\ IS\ 1364-3:2002,\ Table\ 1]$	Pass
End Plate Size (mm)		Square - 116 X 116	Pass
End Plate Thickness (mm)		14	Pass

2.4 Plate Washer and Nut Details - Anchor Bolt Inside Column Flange

Check	Required	Provided	Remarks
Plate Washer Size (mm)		$Square - 58X58$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	**Pass**
Plate Washer Thickness (mm)		$t_w = 8.5$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	**Pass**
Plate Washer Hole Diameter (mm)		$d_h = 33$ $[Ref.\ IS\ 6649:1985,\ Table\ 2]$	**Pass**
Nut (hexagon) Thickness (mm)		$t_n = 18.0$ $[Ref.\ IS\ 1364-3:2002,\ Table\ 1]$	**Pass**
End Plate Size (mm)		Square - 116 X 116	**Pass**
End Plate Thickness (mm)		14	**Pass**

2.5 Anchor Bolt Summary - Outside Column Flange

Check	Required	Provided	Remarks
Diameter (mm)		30	**Pass**
Number of Bolts		$n_{out} = 4$	**Pass**
Property Class		10.9	**Pass**

2.6 Anchor Bolt Summary - Inside Column Flange

Check	Required	Provided	Remarks
Diameter (mm)		30	**Pass**
Number of Bolts		$n_{in} = 4$	**Pass**
Property Class		8.8	**Pass**

2.7 Detailing Checks - Outside Column Flange

Check	Required	Provided	Remarks
Min. End Distance (mm)	$e_{min} = 1.5\ d_0$ $= 1.5 \times 38.0$ $= 57.0$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.2.4.2]$	90	**Pass**
Max. End Distance (mm)	$e_{max} = 12\ t\ \varepsilon;\ \varepsilon = \sqrt{\frac{250}{f_y}}$ $e_1 = 12 \times 56 \times \sqrt{\frac{250}{280}} = 634.98$ $e_2 = 12 \times 56 \times \sqrt{\frac{250}{280}} = 634.98$ $e_{max} = min(e_1, e_2) = 634.98$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.2.4.3]$	90	**Pass**
Min. Edge Distance (mm)	$e'_{min} = 1.5\ d_0$ $= 1.5 \times 38.0$ $= 57.0$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.2.4.2]$	90	**Pass**
Max. Edge Distance (mm)	$e'_{max} = 12\ t\ \varepsilon;\ \varepsilon = \sqrt{\frac{250}{f_y}}$ $e_1 = 12 \times 56 \times \sqrt{\frac{250}{280}} = 634.98$ $e_2 = 12 \times 56 \times \sqrt{\frac{250}{280}} = 634.98$ $e'_{max} = min(e_1, e_2) = 634.98$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.2.4.3]$	90	**Pass**
Min. Pitch Distance (mm)	N/A	0.0	**N/A**
Max. Pitch Distance (mm)	N/A	0.0	**N/A**

2.8 Base Plate Dimension (L X W)

Check	Required	Provided	Remarks
Length (mm)	$L = D + 2\ (e + e)$ $= 450.0 + 2 \times (90 + 90)$ $= 810.0$ $[Ref.\ based\ on\ detailing\ requirement]$	810	**Pass**
Width (mm)	$W = (0.85B) + 2\ (e' + e')$ $= (0.85 \times 250.0) + 2 \times (90 + 90)$ $= 572.5$ $[Ref.\ based\ on\ detailing\ requirement]$	575	**Pass**

2.9 Base Plate Analysis

Check	Required	Provided	Remarks
Eccentricity - about major axis (mm)		$e_{zz} = \dfrac{M_{u\,zz}}{P_u}$ $= \dfrac{223.66 \times 10^6}{1600.0 \times 10^3}$ $= 139.79$	**OK**
Base Plate Type	$\dfrac{L_{min}}{6} < e_{zz} < \dfrac{L_{min}}{3}$ $\dfrac{810.0}{6} < 139.79 < \dfrac{810.0}{3}$ $135.0 < 139.79 < 270.0$	Case 2: The base plate is mostly under compression/bearing while a small tension force being transferred through the anchor bolts outside column flange on the tension side	**OK**
k1	$k_1 = 3\left(e_{zz} - \dfrac{L}{2}\right)$ $= 3\left(139.79 - \dfrac{810}{2}\right)$ $= -795.63$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ , section\ 3.3]$		**OK**

Check	Required	Provided	Remarks
Total Area of Anchor Bolt - under tension (mm^2)	$A_s = n \times \left(\frac{\pi}{4}\right) d^2$ $= 2 \times \left(\frac{\pi}{4}\right) \times 30^2$ $= 1414.0$		**OK**
Lever Arm - distance between the centre of the column and the C.G of the bolt group under tension (mm)	$f = \left(\frac{L}{2} - e\right)$ $= \left(\frac{810}{2} - 90\right)$ $= 315.0$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ , section\ 3.3]$		**OK**
k2	$k_2 = \frac{6\ n\ A_s}{W} \left(f\ +\ e_{zz}\right)$ $= \frac{6\ \times 8.0\ \times 1414.0}{575} \times \left(315.0\ +\ 139.79\right)$ $= 53682.62$ $Note:\ n\ is\ the\ modular\ ratio$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ , section\ 3.3]$		**OK**
k3	$k_3 = -\ k_2\ \left(\frac{L}{2}\ +\ f\right)$ $= -\ 53682.62\ \left(\frac{810}{2}\ +\ 315.0\right)$ $= -38651486.4$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ , section\ 3.3]$		**OK**
Effective Bearing Length (mm)	$y^3\ +\ k_1\ y^2\ +\ k_2\ y\ +\ k_3 = 0$ $y^3\ - 795.63 \times y^2\ + 53682.62 \times y\ - 38651486.4 = 0$ $y = 790.0$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ , section\ 3.3]$		**OK**

Check	Required	Provided	Remarks
Total Tension Demand (kN)	$P_t = -\ P_u \left[\frac{\frac{L}{2} - \frac{y}{3} - e_{zz}}{\frac{L}{2} - \frac{y}{3} + f}\right]$ $= -\ 1600.0 \times \left[\frac{\frac{810}{2} - \frac{790.0}{3} - 139.79}{\frac{810}{2} - \frac{790.0}{3} + 315.0}\right]$ $= 6.58$ $[Ref.\ Design\ of\ Welded\ Structures$ $-\ Omer\ W\ Blodgett\ ,section\ 3.3]$		**OK**
Critical Section - compression side (mm)	$y_{critical} = \frac{L - 0.95D}{2}$ $= \frac{810 - (0.95 \times 450.0)}{2}$ $= 191.25$ $y\ >\ y_{critical}\quad (790.0\ >\ 191.25)$ $Therefore,\ y_{critical} = 191.25$ $Note : The\ critical\ section\ lies\ at$ $0.95D\ of\ the\ column\ section$		**OK**
Bending Moment - at critical section (due to bearing stress) $(N - mm)$	$M_{critical1} = 0.45 f_{ck} W y_{critical} \times \left(\frac{y_{critical}}{2}\right)$ $= 0.45 \times 25.0 \times 575 \times 191.25 \times \left(\frac{191.25}{2}\right)$ $= 118.3 \times 10^6$		**OK**
Lever Arm - distance between center of the flange and bolt group (tension side) (mm)	$l = \frac{L}{2} - \frac{D}{2} + \frac{T}{2} - e$ $= \frac{810}{2} - \frac{450.0}{2} + \frac{13.7}{2} - 90$ $= 96.85$		**OK**
Bending Moment - at critical section (due to tension in the anchor bolts) $(N - mm)$	$M_{critical2} = P_t\ l$ $= 6.58 \times 1000 \times 96.85$ $= 637.27 \times 10^3$		**OK**

Check	Required	Provided	Remarks
Maximum Bending Moment $(N-mm)$	$M_{critical} = max\left(M_{critical1},\ M_{critical2}\right)$ $= max\left(118.3\times10^6,\ 637.27\times10^3\right)$ $= 118.3\times10^6$	Bending of the base plate is governed by the bearing stress caused by the footing	OK
Moment Capacity of Base Plate	$z_{eplate} = \frac{W{t_p}^2}{6}$ $M_{dplate} = 1.5 z_{eplate} f_{y_p}\ /\ \gamma_{m0}$ $= \frac{1.5\left(\frac{W\times t_p^2}{6}\right)f_{y_p}}{\gamma_{m0}}$ $[Ref.\ IS\ 800:2007,\ Cl.8.2.1.2]$		OK
Thickness of Base Plate (mm)	$(T,\ t) < t_p \leq 120$ $(13.7,\ 9.8) < t_p \leq 120$	$M_{dplate} = M_{critical}$ $t_p = \left[\frac{4\ M_{critical}}{W\ (f_{y_p}/\gamma_{m0})}\right]^{0.5}$ $t_p = \left[\frac{4\ \times118.3\times10^6}{575\times\ (300/1.1)}\right]^{0.5}$ $= 54.93$ $= 56$	Pass
Maximum Bearing Stress on Footing (N/mm^2)	$\sigma_{c_{allowable}} = \sigma_{br}$ $= 11.25$	$\sigma_{c_{max}} = \frac{P_t\ y}{A_s\ n\left(\frac{L}{2}-y+f\right)}$ $= \frac{6.58\times10^3\times790.0}{1414.0\times8.0\times\left\|\left(\frac{810}{2}-790.0+315.0\right)\right\|}$ $= 6.56$	Pass

2.10 Anchor Bolt Design - Outside Column Flange

Check	Required	Provided	Remarks
Shear Capacity (kN)		$V_{dsb} = \frac{f_{ub}\ n_n\ A_{nb}}{\sqrt{3}\ \gamma_{mb}}$ $= \frac{1040.0\times1\times561}{1000\times\sqrt{3}\ \times\ 1.25}$ $= 269.48$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.3.3]$	OK

Check	Required	Provided	Remarks
Kb		$k_b = min\left(\frac{e}{3d_0}, \frac{f_{ub}}{f_u}, 1.0\right)$ $= min\left(\frac{90}{3 \times 38.0}, \frac{1040.0}{440.0}, 1.0\right)$ $= min(0.79, 2.36, 1.0)$ $= 0.79$ $[Ref\ IS\ 800:2007,\ Cl.\ 10.3.4]$	**OK**
Bearing Capacity (kN)		$V_{dpb} = \frac{2.5\ k_b\ d\ t\ f_u}{\gamma_{mb}}$ $= \frac{2.5 \times 0.79 \times 30 \times 56 \times 440}{1000 \times 1.25}$ $= 1167.94$ $= 0.7 \times 1167.94$ $= 817.56$ $Note:\ The\ bearing\ capacity\ is\ reduced$ $since\ the\ hole\ type\ is\ Over-sized$ $or\ Short-slotted$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.3.4]$	**OK**
Bolt Capacity (kN)		$V_{db} = min\ (V_{dsb}, V_{dpb})$ $= min\ (269.48, 817.56)$ $= 269.48$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.3.2]$	**OK**
Tension Demand - per anchor bolt (kN)	$T_b = \frac{P_t}{n_{out}/2}$ $= \frac{6.58}{4/2}$ $= \frac{6.58}{2}$ $= 3.29$	$T_{db} = 0.90\ f_{ub}\ A_n\ /\ \gamma_{mb}$ $<\ f_{yb}\ A_{sb}\ (\gamma_{mb}\ /\ \gamma_{m0})$ $= min\Big(0.90\ \times 1040.0 \times 561\ /\ 1.25,$ $940.0 \times 707 \times (1.25/1.1)\Big)$ $= min(420.08,\ 755.2)$ $= 420.08$ $[Ref.\ IS\ 800:2007,\ Cl.\ 10.3.5]$	**Pass**

Check	Required	Provided	Remarks
Anchor Length - above concrete footing (mm)		$l_1 = t_g + t_p + t_w + t_n + 20$ $= 50 + 56 + 8.5 + 25.6 + 20$ $= 160.1$	**Pass**
Anchor Length - below concrete footing (mm)		$l_2 = \left[\frac{T_{db}}{15.5\sqrt{f_{ck}}}\right]^{0.67}$ $= \left[\frac{420.08 \times 10^3}{15.5 \times \sqrt{25.0}}\right]^{0.67}$ $= 317.54$ $= 320$ $= max(320,\ 320)$ $= 320$ $= 320 + t_n + 20$ $= 320 + 25.6 + 20$ $= 365.6$ $[Reference:\ Design\ of\ Steel\ Structures$ $-\ N.Subramanian,\ (2019\ edition),$ $Chapter\ 15,\ Example\ 15.5]$	**Pass**
Anchor Length - total (mm)	$320 \leq\ l_a\ \leq 2000$ $[Reference:\ IS\ 5624:1993,\ Table\ 1]$	$l_a = l_1 + l_2$ $= 160.1 + 365.6$ $= 525.7$	**Pass**

2.11 Anchor Bolt Design - Inside Column Flange

Check	Required	Provided	Remarks
Shear Capacity (kN)	The bolts are not designed to carry shear force	N/A	**N/A**
Bearing Capacity (kN)	The bolts are not designed to carry shear force	N/A	**N/A**
Bolt Capacity (kN)	N/A	N/A	**N/A**
Tension Demand (kN)	$P_{uplift} = 0.0$		**OK**

Check	Required	Provided	Remarks
Tension Capacity (kN)		$T_{db} = 0.90\ f_{ub}\ A_n\ /\ \gamma_{mb}$ $<\ f_{yb}\ A_{sb}\ (\gamma_{mb}\ /\ \gamma_{m0})$ $= min\Big(0.90\ \times 830.0 \times 245\ /\ 1.25,$ $660.0 \times 314 \times (1.25/1.1)\Big)$ $= min(146.41,\ 235.5)$ $= 146.41$ $[Ref.\ IS\ 800 : 2007,\ Cl.\ 10.3.5]$	**OK**
Anchor Bolts Required (kN)	$n_{in} = \dfrac{P_{uplift}}{T_{db}}$ $= \dfrac{0.0}{146.41}$ $= 0.0$	4	
Anchor Length - above concrete footing (mm)		$l_1 = t_g\ +\ t_p\ +\ t_w\ +\ t_n\ +\ 20$ $= 50\ +\ 56\ +\ 8.5\ +\ 18.0\ +\ 20$ $= 160.1$	**Pass**
Anchor Length - below concrete footing (mm)		$l_2 = \left[\dfrac{T_{db}}{15.5\sqrt{f_{ck}}}\right]^{0.67}$ $= \left[\dfrac{146.41 \times 10^3}{15.5 \times \sqrt{25.0}}\right]^{0.67}$ $= 200$ $= 200$ $= max(200,\ 200)$ $= 200$ $= 200 + t_n + 20$ $= 200 + 18.0 + 20$ $= 365.6$ $[Reference:\ Design\ of\ Steel\ Structures$ $-\ N.Subramanian,\ (2019\ edition),$ $Chapter\ 15,\ Example\ 15.5]$	**Pass**

Check	Required	Provided	Remarks
Anchor Length - total (mm)	$200 \leq\ l_a\ \leq 800$ $[Reference:\ IS\ 5624:1993,\ Table\ 1]$	$l_a = l_1\ +\ l_2$ $= 160.1\ +\ 365.6$ $= 390.5$	**Pass**

2.12 Stiffener Design - Along Column Flange

Check	Required	Provided	Remarks
Length of Stiffener (mm)		$L_{stf} = \frac{W-B}{2}$ $= \frac{575-250.0}{2}$ $= 162.5$ $[Ref.\ based\ on\ detailing\ requirement]$	**OK**
Height of Stiffener (mm)		$H_{stf} = L_{stf}\ +\ 50$ $= 162.5\ +\ 50$ $= 212.5$	**OK**
Thickness of Stiffener (mm)	$t_{stf} = \left(\frac{L_{stf}}{13.6 \times \epsilon_{st}}\right) \geq T$ $= max\left(\left(\frac{162.5}{13.6 \times 0.91}\right),\ 13.7\right)$ $= max(13.13, 13.7)$ $Note:\ The\ stiffener\ is\ assumed$ $to\ be\ semi-compact$ $[Ref.\ IS\ 800:2007,\ Table\ 2]$	22	**Pass**
Stress (average) at Stiffener (N/mm^2)	$= \sigma_{callowable}$ $= 11.25$	$Since,\ y > y_{critical}\quad (790.0 > 191.25)$ $\sigma_{stf} = \frac{\sigma_{cmax}}{2}$ $= \frac{6.56}{2}$ $= 3.28$	**Pass**

Check	Required	Provided	Remarks
Shear on Stiffener (kN)	$V_{st\,f} = \sigma_{st\,f}\left(y \times L_{st\,f}\right)$ $= 3.28 \times \left(790.0 \times 162.5\right) \times 10^{-3}$ $= 421.07$	$V_{d\,f} = \frac{A_{vg}\ f_{y\,st}}{\sqrt{3}\ \gamma_{m0}}$ $= \frac{(H_{st\,f} \times t_{st\,f}) f_{y\,st}}{\sqrt{3}\ \gamma_{m0}}$ $= \frac{(212.5 \times 22) \times 300}{\sqrt{3} \times 1.1 \times 10^{3}}$ $= 736.122$ $Note:\ Stiffener\ is\ not\ restricted\ to\ low\ shear$ $[Ref.\ IS\ 800:2007\ (Cl.\ 8.4.1)$	Pass
Section Modulus of the Stiffener (mm^3)		$z_{e\,st} = 165.57 \times 10^{3}$	OK
Moment on Stiffener (kNm)	$M_{st\,f} = \sigma_{st\,f}\left(y \times \frac{{L_{st\,f}}^{2}}{2}\right)$ $= 3.28 \times \left(790.0 \times \frac{162.5^{2}}{2}\right) \times 10^{-6}$ $= 34.212$	$M_{d\,f} = \frac{\beta_b\ z_{e\,st}\ f_{y\,st}}{\gamma_{m0}}$ $= \frac{1 \times\ z_{e\,st}\ f_{y\,st}}{\gamma_{m0}} \quad (\beta_b = 1)$ $= \frac{1 \times\ 165.57 \times 10^{3} \times 300}{1.1 \times 10^{6}}$ $= 45.156$ $[Ref.\ IS\ 800:2007\ (Cl.\ 8.2.1.2)$	Pass
Weld Size (mm)	10	12	Pass

2.13 Stiffener Design - Along Column Web

Check	Required	Provided	Remarks
Length of Stiffener (mm)		$L_{st\,w} = \frac{L - D}{2}$ $= \frac{810 - 450.0}{2}$ $= 180.0$ $[Ref.\ based\ on\ detailing\ requirement$	OK
Height of Stiffener (mm)		$H_{st\,w} = L_{st\,w}\ +\ 50$ $= 180.0\ +\ 50$ $= 230.0$	OK

Check	Required	Provided	Remarks
Thickness of Stiffener (mm)	$t_{st_w} = \left(\frac{L_{st_w}}{13.6 \times \epsilon_{st}}\right) \geq t$ $= max\left(\left(\frac{180.0}{13.6 \times 0.91}\right), 9.8\right)$ $= max(14.54, 9.8)$ $[Ref.\ IS\ 800:2007\ , Table\ 2]$	16	Pass
Stress (average) at Stiffener (mm)	$= \sigma_{callowable}$ $= 11.25$	$\sigma_{st_w} = \frac{\sigma_{cmax} + \sigma_{crt}}{2}$ $= \frac{6.56 + 4.98}{2}$ $= 5.77$	Pass
Shear on Stiffener (kN)	$V_{st_w} = \sigma_{st_w}\left(B\ L_{st_w}\right)$ $= 5.77 \times \left(250.0 \times 180.0\right) \times 10^{-3}$ $= 259.65$	$V_{d_w} = \frac{A_{vg}\ f_{y_{st}}}{\sqrt{3}\ \gamma_{m0}}$ $= \frac{(H_{st_w} \times t_{st_w}) f_{y_{st}}}{\sqrt{3}\ \gamma_{m0}}$ $= \frac{(230.0 \times 16) \times 300}{\sqrt{3} \times 1.1 \times 10^{3}}$ $= 579.45$ $Note:\ Stiffener\ is\ not\ restricted\ to\ low\ shear$ $[Ref.\ IS\ 800:2007\ (Cl.\ 8.4.1)$	Pass
Section Modulus of the Stiffener (mm^3)		$z_{e_{st}} = 141.07 \times 10^{3}$	OK
Moment on Stiffener (kNm)	$M_{st_w} = \left(\sigma_{crt} \times B \times \frac{{L_{st_w}}^2}{2}\right) +$ $\left(\left(\sigma_{cmax} - \sigma_{crt}\right) \times B \times \frac{{L_{st_w}}^2}{3}\right)$ $= \left[\left(4.98 \times 250.0 \times \frac{180.0^2}{2}\right) +\right.$ $\left.\left(\left(6.56 - 4.98\right) \times 250.0 \times \frac{180.0^2}{3}\right)\right] \times 10^{-6}$ $= 24.435$	$M_{d_w} = \frac{\beta_b\ z_{e_{st}}\ f_{y_{st}}}{\gamma_{m0}}$ $= \frac{1 \times\ z_{e_{st}}\ f_{y_{st}}}{\gamma_{m0}} \quad (\beta_b = 1)$ $= \frac{1 \times\ 141.07 \times 10^{3} \times 300}{1.1 \times 10^{6}}$ $= 38.473$ $[Ref.\ IS\ 800:2007\ (Cl.\ 8.2.1.2)$	Pass
Weld Size (mm)	10	12	Pass

2.14 Stiffener Design - Across Column Web

Check	Required	Provided	Remarks
Length of Stiffener (mm)		$L_{staw} = max\ (L_{stf},\ L_{stw})$ $\leq \frac{W-t}{2}$ $= max\ (162.5,\ 180.0)$ $= 180.0$	**Pass**
Height of Stiffener (mm)		$H_{staw} = L_{staw}\ +\ 50$ $= 180.0\ +\ 50$ $= 230.0$	**Pass**
Thickness of Stiffener (mm)	$t_{staw} = \left(\frac{L_{staw}}{13.6 \times \epsilon_{st}}\right) \geq t$ $= max\left(\left(\frac{180.0}{13.6 \times 0.91}\right),\ 9.8\right)$ $= max(14.54, 9.8)$ $[Ref.\ IS\ 800:2007,\ Table\ 2]$	16	**Pass**
Weld Size (mm)	10	8	**Pass**

2.15 Shear Design

Check	Required	Provided	Remarks
Shear Resistance (kN)		$V_r = P_u \times \mu$ $= 1600.0 \times 0.45$ $= 720.0$	**OK**
Shear Key Requirement - along column depth	$V_1 = 80.0\ \ kN$	$V_1 \leq V_r$ $80.0 \leq 720.0$	**Shear key not required**
Shear Key Requirement - along column width	$V_2 = 23.0\ \ kN$	$V_2 \leq V_r$ $23.0 \leq 720.0$	**Shear key not required**

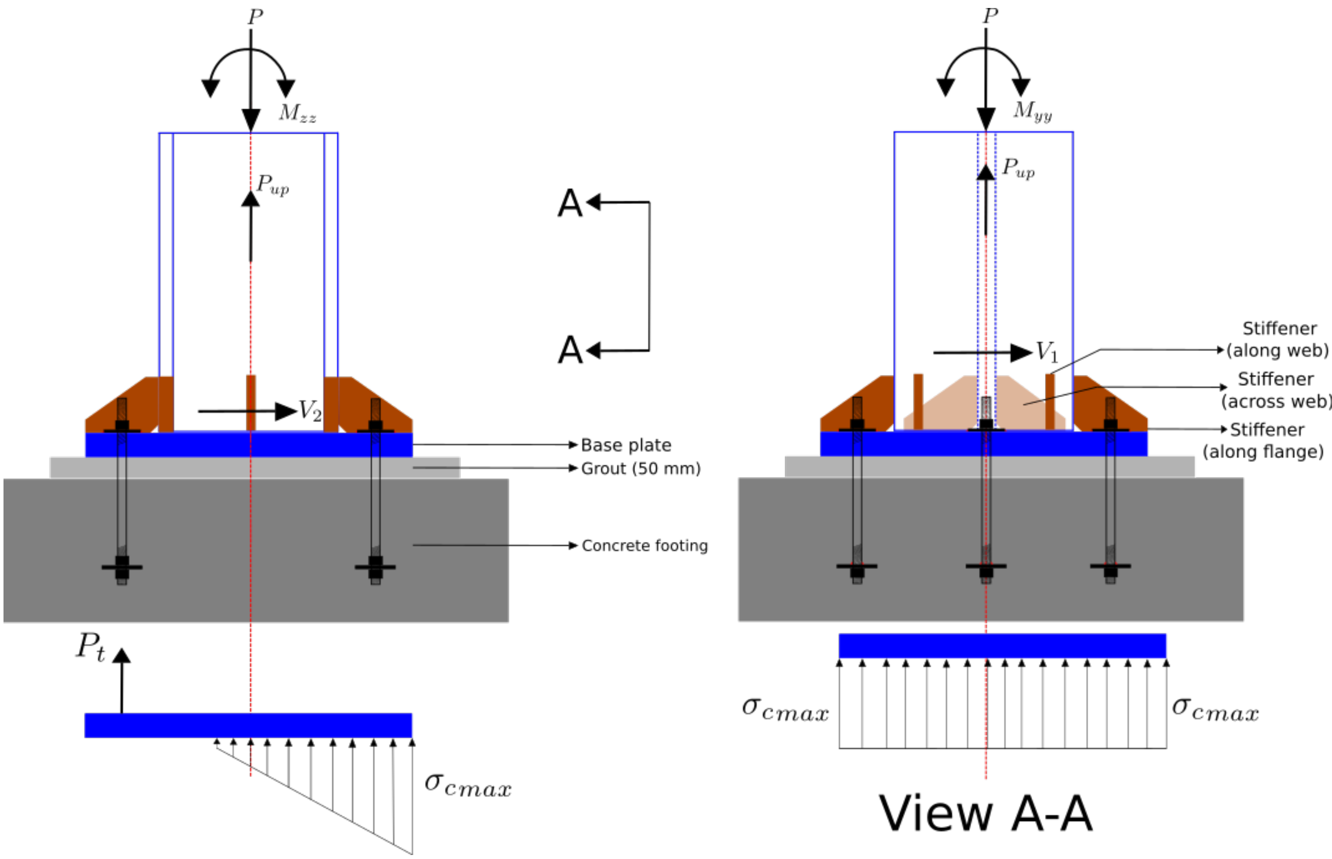


Figure 1: Typical Base Plate Details

3 2D Drawings (Typical)

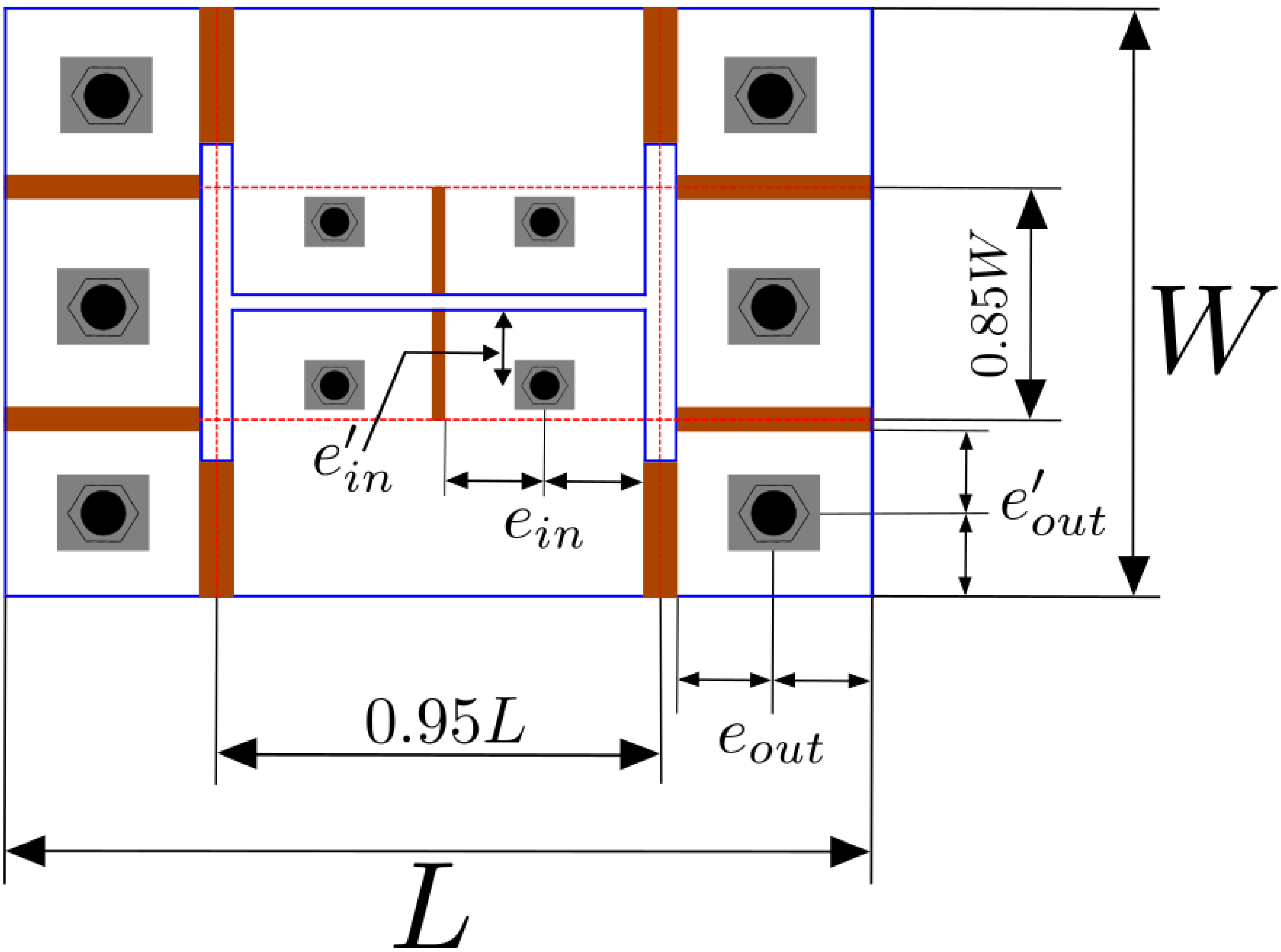


Figure 2: Typical Base Plate Detailing

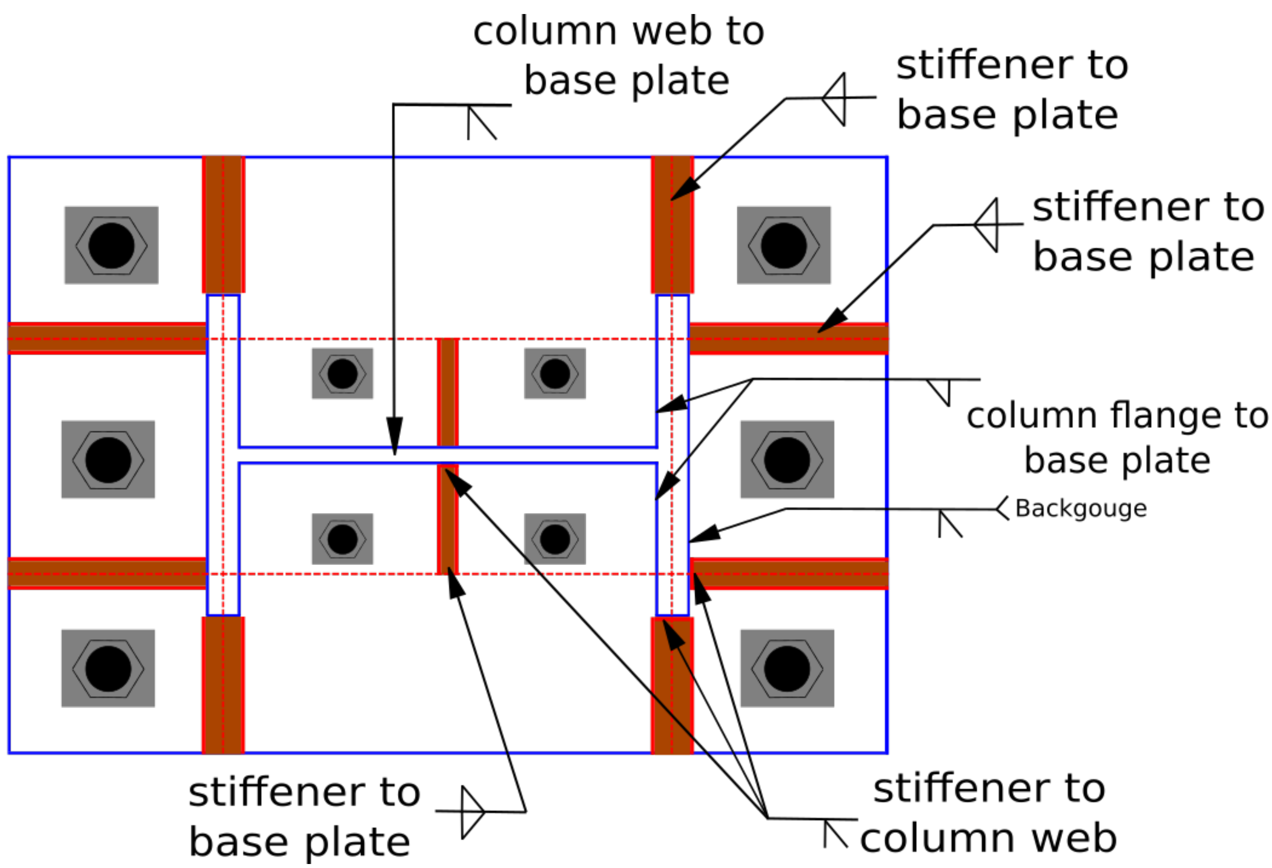


Figure 3: Typical Weld Details

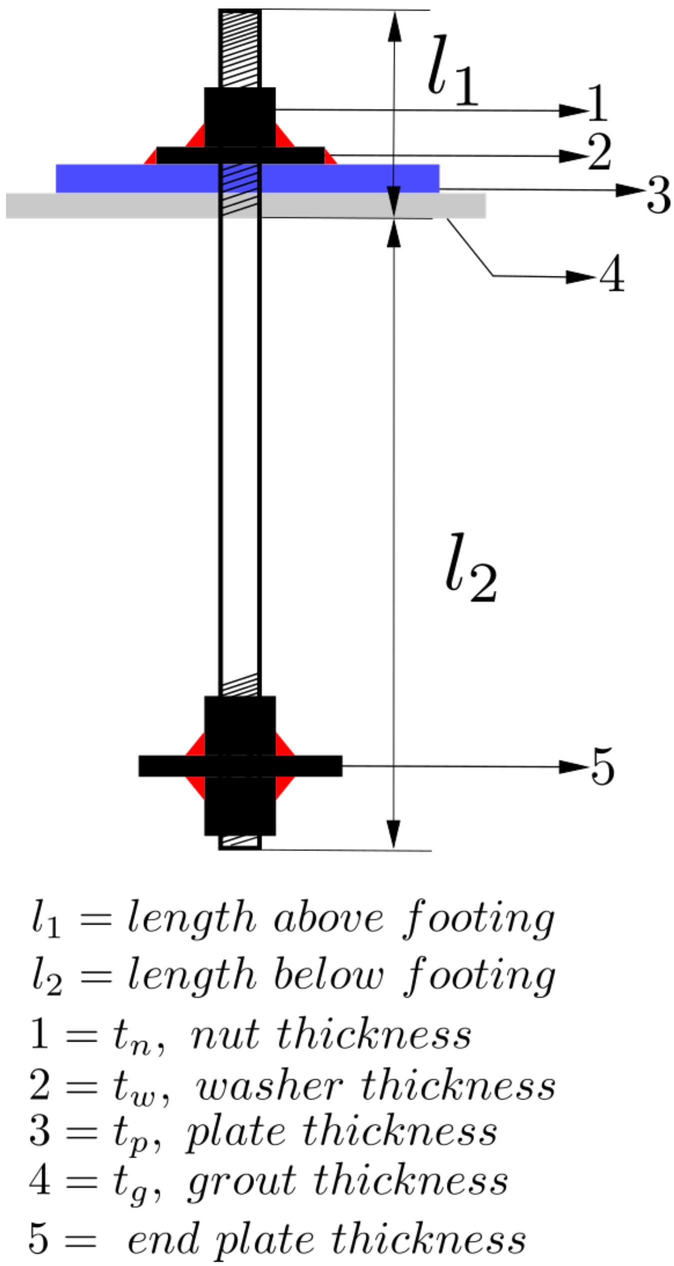
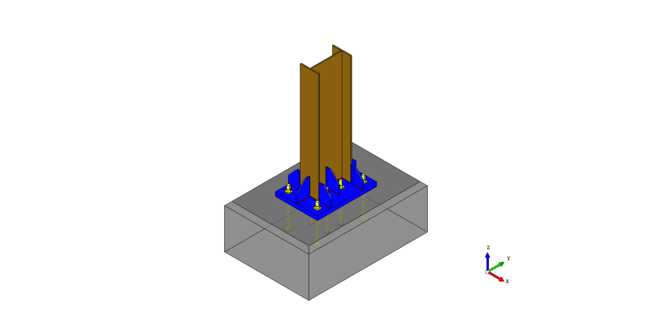
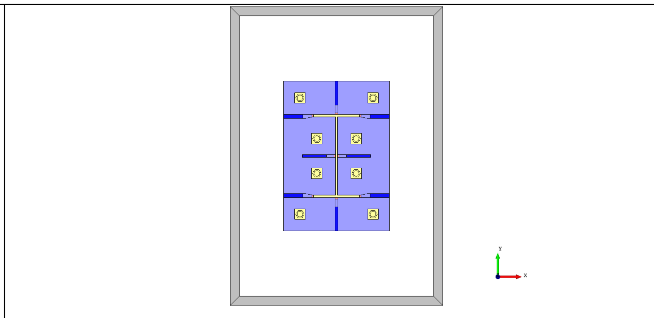


Figure 4: Typical Anchor Bolt Details

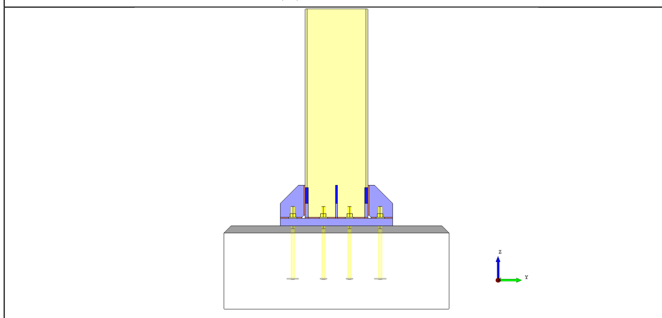
4 3D Views



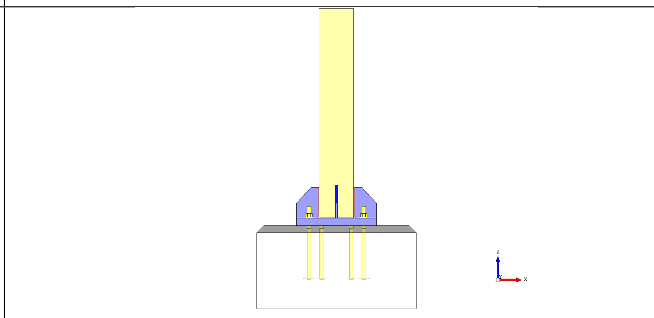
(a) 3D View



(b) Top View



(c) Side View



(d) Front View

5 Design Log

2020-12-18 01:44:52 - Osdag - WARNING - The Load(s) defined is/are less than the minimum recommended value [Ref. IS 800:2007, Cl.10.7].
2020-12-18 01:44:52 - Osdag - WARNING - [Minimum Factored Load] The external factored bending moment (447.32 kNm) is less than 0.5 times the plastic moment capacity of the column (237.27 kNm)
2020-12-18 01:44:52 - Osdag - INFO - The minimum factored bending moment should be at least 0.5 times the plastic moment capacity of the beam to qualify the connection as rigid connection
2020-12-18 01:44:52 - Osdag - INFO - The value of load(s) is/are set at minimum recommended value as per Cl.10.7
2020-12-18 01:44:52 - Osdag - INFO - Designing the connection for a factored moment of 223.66 kNm
2020-12-18 01:44:52 - Osdag - INFO - [Base Plate Type] The value of eccentricity about the major axis is 138 mm
2020-12-18 01:44:52 - Osdag - INFO - Eccentricity is greater than 135.0 (L/6) mm but less than 270.0 (L/3) mm
2020-12-18 01:44:52 - Osdag - INFO - Case 2: A larger part of the base plate is under compression/bearing with a small to moderate tension/uplift force being transferred through the anchor bolts outside column flange on the tension side
2020-12-18 01:44:52 - Osdag - INFO - [Base Plate Type] The value of eccentricity about the major axis is 138 mm
2020-12-18 01:44:52 - Osdag - INFO - Eccentricity is greater than 135.0 (L/6) mm but less than 270.0 (L/3) mm
2020-12-18 01:44:52 - Osdag - INFO - Case 2: A larger part of the base plate is under compression/bearing with a small to moderate

tension/uplift force being transferred through the anchor bolts outside column flange on the tension side
2020-12-18 01:44:52 - Osdag - INFO - [Minor Axis Moment] The value of eccentricity about the minor axis is 28 mm
2020-12-18 01:44:52 - Osdag - INFO - Eccentricity is less than 95.83 mm (W/6)
2020-12-18 01:44:52 - Osdag - INFO - Case 1: The base plate is purely under compression/bearing over it's width, thus there is no requirement of anchor bolts along the width of the column section
2020-12-18 01:44:52 - Osdag - INFO - [Design for Shear] The shear resistance of the base plate assembly due to the friction between the base plate and the grout/concrete material is 720.0 kN
2020-12-18 01:44:52 - Osdag - INFO - The horizontal shear force - 80.0 kN, is less than the shear resistance of the base plate
2020-12-18 01:44:52 - Osdag - INFO - Shear key is not required
2020-12-18 01:44:52 - Osdag - INFO - [Design for Shear] The shear resistance of the base plate assembly due to the friction between the base plate and the grout/concrete material is 720.0 kN
2020-12-18 01:44:52 - Osdag - INFO - The horizontal shear force - 80.0 kN, is less than the shear resistance of the base plate
2020-12-18 01:44:52 - Osdag - INFO - Shear key is not required
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] The length of the anchor bolt is computed assuming the anchor bolt is casted in-situ during the erection of the column.
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] The recommended range for the length of the anchor bolt of thread size 30 mm is as follows:
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] Minimum length = 320 mm, Maximum length = 2000 mm.
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] The provided length of the anchor bolt is 525.7 mm
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt] Designer/Erector should provide adequate anchorage depending on the availability of standard lengths and sizes, satisfying the recommended range
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] Reference: IS 5624:1993, Table 1
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] The recommended range for the length of the anchor bolt of thread size 20 mm is as follows:
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] Minimum length = 200 mm, Maximum length = 800 mm.
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] The provided length of the anchor bolt is 390.5 mm
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt] Designer/Erector should provide adequate anchorage depending on the availability of standard lengths and sizes, satisfying the recommended range
2020-12-18 01:44:52 - Osdag - INFO - [Anchor Bolt Length] Reference: IS 5624:1993, Table 1
2020-12-18 01:44:52 - Osdag - WARNING - [Shear Check - Stiffener] The stiffener along the flange fails the shear check
2020-12-18 01:44:52 - Osdag - WARNING - The shear demand on the stiffener (421.07 kN) exceeds 60% of it's capacity (281.06 kN)
2020-12-18 01:44:52 - Osdag - INFO - Increasing the thickness of the stiffener and re-checking against shear demand