Osoconn

Validation Record for

MC001AM10

Extended End Plate Seismic Moment Connection

(March 27, 2025)

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1 Introduction

Osoconn is a free and open source connection design application. The Osoconn project is a personal project developed by Roshn Noronha for educational purposes and licensed under the MIT Open Source license. For more information visit `https://osoconn.com`.

1.1 Purpose and scope

The purpose of this document is to validate the results of the connection code MC001AM10 for the Osoconn project.

1.2 Methodology

To validate the results of the program a set of sample calculations are prepared and the results are compared with the output from the program. If the results obtained are equal within a tolerance of one percent, the validation is deemed successful.

The connection code MC001AM10 refers to the extended end plate seismic moment connection, and the design of this connection type is checked against the requirements of AISC 360-2010 [1]. The detailed calculation and a summary of the comparison with the program output is provided in section 2. The full output of the program is provided in section 3.

To minimize the chance of errors the selected validation problems tries to cover as many different options and connections configurations available in the program as possible. However, while every attempt is made to ensure the accuracy of the program, it should be noted that, not every aspect of the program can be tested,and the user shall independently verify the output of the program before using it.

References

[1] AISC. *Specification for Structural Steel Buildings*. 360. American Institute of Steel Construction, Chicago, IL, 2010.

2 Validation Calculation

2.1 Executive summary

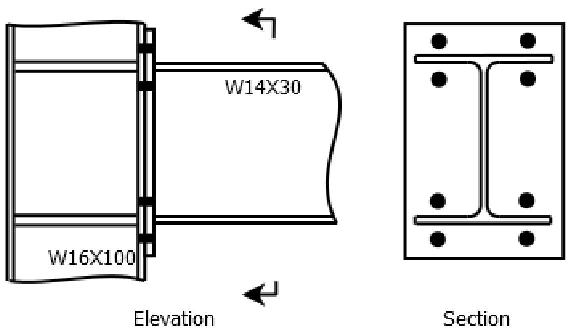
Table 1: Executive Summary

	Result
Validation problem 1	OK
Validation problem 2	OK
Validation problem 3	OK
Validation problem 4	OK
Validation problem 5	OK
Validation problem 6	OK

2.2 Validation Problem 1

Problem Statement

Design a 4-bolt unstiffened extended end-plate moment connection for a W14X30 beam framing into the flange of a W16X100 column. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 35kip, and the axial force in the column is 150kip. The beam, column and plates are of grade ASTM A36. The bolts are ASTM A3125 A325.



Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 36\ ksi$
Tensile strength	$F_{up} := 58\ ksi$
Material grade of beam	ASTM A36
Yield strength	$F_{yb} := 36\ ksi$
Tensile strength	$F_{ub} := 58\ ksi$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.5$
Material grade of column	ASTM A36
Yield strength	$F_{yc} := 36\ ksi$
Tensile strength	$F_{uc} := 58\ ksi$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 70\ ksi$
Material specification for bolts	ASTM 3125 A325
Tensile strength	$F_{nt} := 90\ ksi$
Shear strength	$F_{nv} := 54\ ksi$
Young's modulus for steel	$E := 29000\ ksi$

Design Forces

Shear force in beam	$V_u := 35\ kip$
Axial force in column	$P_{uc} := 150\ kip$

Connection Geometry

Beam section — W14X30

Section depth — $d_{xb} := 13.8\ in$

Flange width — $b_{fb} := 6.73\ in$

Flange thickness — $t_{fb} := 0.385\ in$

Web thickness — $t_{wb} := 0.27\ in$

Design distance from outer face to fillet edge — $k_{des} := 0.785\ in$

Detailing distance from outer face to fillet edge — $k_{det} := 1.125\ in$

Plastic section modulus — $Z_x := 47.3\ in^3$

Column section — W16X100

Section depth — $d_{xc} := 17\ in$

Flange width — $b_{fc} := 10.4\ in$

Flange thickness — $t_{fc} := 0.985\ in$

Web thickness — $t_{wc} := 0.585\ in$

Cross section area of column — $A_c := 29.4\ in^2$

Design dist form outer face to fillet edge — $k_c := 1.39\ in$

End-plate width — $B := 12\ in$

End-plate height — $H := 22\ in$

End-plate thickness — $t := 1\ in$

Bolt diameter — $d_b := 1.125\ in$

Bolt hole diameter — $d_{bh} := 1.188\ in$

Inner bolt pitch — $p_i := 1.75\ in$

Outer bolt pitch — $p_o := 1.75\ in$

Bolt gage — $g := 5.5\ in$

Web weld thickness — $w_1 := 0.25\ in$

Flange weld reinforcement — $w_2 := 0.16\ in$

Column stiffener thickness — $t_{st} := 1\ in$

Width of column stiffener — $B_{st} := 4\ in$

Column stiffener clip dimension — $k_{st} := 1\ in$

Thickness of stiffener to flange weld — $t_{w4} := 0.5\ in$

Thickness of stiffener to web weld — $t_{w5} := 0.25\ in$

Column extension from top of beam — $a := 3\ in$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 234.135\ kip \cdot ft$$

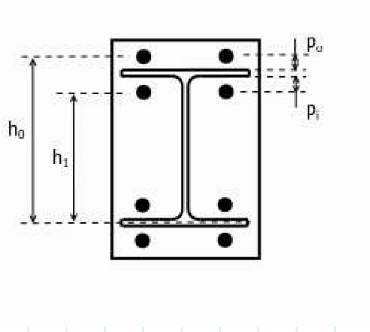
Design moment at face of column

$$M_u := M_p + V_u \cdot \left(min\left(\frac{d_{xb}}{2}, 3 \cdot b_{fb} \right) \right) \qquad M_u = 254.26\ kip \cdot ft$$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}} \qquad P_{uf} = 227.441\ kip$$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$h_0 := d_{xb} - 0.5\ t_{fb} + p_o$ $\qquad h_0 = 15.358\ in$

$h_1 := d_{xb} - t_{fb} - p_i$ $\qquad h_1 = 11.665\ in$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2 \cdot M_u}{\pi \cdot 0.75 \cdot F_{nt} \cdot \sum h}} \qquad d_{b.min} = 1.032\ in$$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b} \qquad I_0 = 0.917$$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot d_b^{\ 2}}{4} \qquad A_b = 0.994\ in^2$$

Nominal strength of bolt is shear

$R_{n.bv} := F_{nv} \cdot A_b$ $\qquad R_{n.bv} = 53.677\ kip$

Interaction ratio in bolt shear

$$I_1 := \frac{V_u}{4 \cdot 0.75 \cdot R_{n.bv}} \qquad I_1 = 0.217$$

Bolt bearing on end plate check

Bolt hole edge to edge distance

$ed_1 := 0.5 \cdot (H - d_{xb}) - p_o - 0.5 \cdot d_{bh}$ $\qquad ed_1 = 1.756\ in$

Bolt hole to hole edge distance

$s_1 := p_i + p_o - d_{bh}$ $\qquad s_1 = 2.312\ in$

Clear distances for bearing calculation

For outer bolt

$L_{co} := min(ed_1, s_1)$ $\qquad L_{co} = 1.756\ in$

For inner bolt

$L_{ci} := s_1$ $\qquad L_{ci} = 2.312\ in$

Nominal strength in bearing

For outer bolts

$R_{n.bbpo} := 2 \cdot min\left(1.2 \cdot L_{co} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $\quad R_{n.bbpo} = 244.435\ kip$

For inner bolts

$R_{n.bbpi} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $\quad R_{n.bbpi} = 313.2\ kip$

Net bearing strength

$R_{n.bbp} := R_{n.bbpi} + R_{n.bbpo}$ $\quad R_{n.bbp} = 557.635\ kip$

Interaction ratio in bearing at end plate

$I_{2} := \dfrac{V_u}{0.75 \cdot R_{n.bbp}}$ $\quad I_{2} = 0.084$

Bolt bearing at column flange

Bolt hole edge to edge of column

$ed_2 := a - p_o - 0.5\ d_{bh}$ $\quad ed_2 = 0.656\ in$

Clear distances for bearing calculation

For outer bolt

$L_{co2} := min\left(ed_2, s_1\right)$ $\quad L_{co} = 1.756\ in$

Nominal strength in bearing

For outer bolts

$R_{n.bbco} := 2 \cdot min\left(1.2 \cdot L_{co2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $\quad R_{n.bbco} = 89.945\ kip$

For inner bolts

$R_{n.bbci} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $\quad R_{n.bbci} = 308.502\ kip$

Net bearing strength

$R_{n.bbc} := R_{n.bbci} + R_{n.bbco}$ $\quad R_{n.bbc} = 398.447\ kip$

Interaction ratio in bearing at end plate

$I_{3} := \dfrac{V_u}{0.75 \cdot R_{n.bbc}}$ $\quad I_{3} = 0.117$

Plate thickness check

Bolt tension strength

$P_t := F_{nt} \cdot A_b$ $\quad P_t = 89.462\ kip$

No-prying moment strength of plate

$M_{np} := 2 \cdot P_t \cdot \sum h$ $\quad M_{np} = 402.913\ kip \cdot ft$

Dimension

$s := \dfrac{1}{2} \cdot \sqrt{B \cdot g}$ $\quad s = 4.062\ in$

$p'_i := min\left(p_i, s\right)$ $\quad p'_i = 1.75\ in$

End plate yield line parameter

$$Y_p := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{p_i} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_o} \right) - \frac{1}{2} \right) + \frac{2}{g} \left(h_1 \cdot (p_i + s) \right) \qquad Y_p = 131.532 \; in$$

Required end plate thickness

$$t_{p.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yp} \cdot Y_p}} \qquad t_{p.min} = 0.972 \; in$$

Interaction ratio for plate thickness

$$I_4 := \frac{t_{p.min}}{t} \qquad I_4 = 0.972$$

Plate shear yielding check

Nominal strength of plate in shear yielding

$$R_{n.py} := 0.6 \cdot F_{yp} \cdot B \cdot t \qquad R_{n.py} = 259.2 \; kip$$

Interaction ratio in shear yielding

$$I_5 := \frac{P_{uf}}{2 \cdot 1.0 \cdot R_{n.py}} \qquad I_5 = 0.439$$

Plate shear rupture check

Net area of plate in shear

$$A_n := \left(B - 2 \cdot d_{bh} \right) \cdot t \qquad A_n = 9.624 \; in^2$$

Nominal strength of plate in shear rupture

$$R_{n.pr} := 0.6 \cdot F_{up} \cdot A_n \qquad R_{n.pr} = 334.915 \; kip$$

Interaction ratio in shear rupture

$$I_6 := \frac{P_{uf}}{2 \cdot 0.75 \cdot R_{n.pr}} \qquad I_6 = 0.453$$

Beam web to plate weld tension check

Required strength of weld in tension

$$f_{wt} := F_{yb} \cdot t_{wb} \qquad f_{wt} = 9.72 \; \frac{kip}{in}$$

Nominal strength of weld in tension

$$f_{n.wt} := 2 \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}} \cdot 1.5 \qquad f_{n.wt} = 22.274 \; \frac{kip}{in}$$

Interaction ratio for weld in tension

$$I_7 := \frac{f_{wt}}{0.75 \cdot f_{n.wt}} \qquad I_7 = 0.582$$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws} := d_{xb} - t_{fb} - p_i - 6 \; in - k_{des} \qquad l_{ws} = 4.88 \; in$$

Nominal strenght of weld in shear

$$V_{n.ws} := 2 \cdot l_{ws} \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}} \qquad V_{n.ws} = 72.464\ kip$$

Interaction ratio for weld in shear

$$I_8 := \frac{V_u}{0.75 \cdot V_{n.ws}} \qquad I_8 = 0.644$$

Column flange flexural yielding

Dimensions

$$s_c := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} \qquad s_c = 3.782\ in$$

$$p_{si} := p_i + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{si} = 1.443\ in$$

$$p_{so} := p_o + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{so} = 1.443\ in$$

Yield line parameter for column flange

$$Y_c := \frac{b_{fc}}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s_c} + \frac{1}{p_{si}} \right) + h_0 \cdot \left(\frac{1}{s_c} + \frac{1}{p_{so}} \right) \right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(s_c + p_{si} \right) + h_0 \cdot \left(s_c + p_{so} \right) \right)$$

$$Y_c = 185.904\ in$$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yc} \cdot Y_c}} \qquad t_{fc.min} = 0.817\ in$$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}} \qquad I_9 = 0.83$$

Column web yielding strength

Web yielding factor

$$C_t := \text{if} \left(a < d_{xc}, 0.5, 1 \right) \qquad C_t = 0.5$$

Bearing length of beam flange

$$N := t_{fb} + 2 \cdot w_2 \qquad N = 0.705\ in$$

Nominal strength in web yielding

$$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t \right) + N \right) \cdot F_{yc} \cdot t_{wc} \qquad R_{n.wy} = 123.728\ kip$$

Interaction ratio in web yielding

$$I_{10} := \frac{P_{uf}}{1.0 \cdot R_{n.wy}} \qquad I_{10} := 0$$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot {t_{wc}}^2 \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}} \right) \cdot \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2}:=0.4\cdot t_{wc}^{\;2}\cdot\left(1+3\cdot\left(\frac{N}{d_{xc}}\right)\cdot\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right)\cdot\sqrt{\frac{E\cdot F_{yc}\cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3}:=0.4\cdot t_{wc}^{\;2}\cdot\left(1+\left(\frac{4\cdot N}{d_{xc}}-0.2\right)\cdot\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right)\cdot\sqrt{\frac{E\cdot F_{yc}\cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc}:=\text{if}\left(\left(a+0.5\cdot t_{fb}\right)\geq 0.5\cdot d_{xc},R_{n.wc1},\text{if}\left(\left(N\div d_{xc}\right)<0.2,R_{n.wc2},R_{n.wc3}\right)\right)$$

$R_{n.wc}=191.829\ \textit{kip}$

Interaction ratio for web crippling

$$I_{11}:=\frac{P_{uf}}{0.75\cdot R_{n.wc}}$$ $I_{11}:=0$

Column panel shear strength

Axial yield strength of column

$$P_y:=F_{yc}\cdot A_c$$ $P_y=\left(1.058\cdot 10^3\right)\ \textit{kip}$

Panel zone shear strength

$$R_{n.pz1}:=0.6\cdot F_{yc}\cdot d_{xc}\cdot t_{wc}\cdot\left(1+\frac{3\cdot b_{fc}\cdot t_{fc}^{\;2}}{d_{xb}\cdot d_{xc}\cdot t_{wc}}\right)$$

$$R_{n.pz2}:=0.6\cdot F_{yc}\cdot d_{xc}\cdot t_{wc}\cdot\left(1+\frac{3\cdot b_{fc}\cdot t_{fc}^{\;2}}{d_{xb}\cdot d_{xc}\cdot t_{wc}}\right)\cdot\left(1.9-\frac{1.2\cdot P_{uc}}{P_y}\right)$$

$$R_{n.pz}:=\text{if}\left(P_{uc}\leq 0.75\cdot P_y,R_{n.pz1},R_{n.pz2}\right)$$ $R_{n.pz}=262.193\ \textit{kip}$

Interaction ratio in panel zone shear

$$I_{12}:=\frac{P_{uf}}{0.9\cdot R_{n.pz}}$$ $I_{12}=0.964$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min}:=\frac{d_{xb}-t_{fb}+d_{xc}-2\cdot t_{fc}}{90}$$ $t_{wc.min}=0.316\ \textit{in}$

Interaction ratio in web shear buckling

$$I_{13}:=\frac{t_{wc.min}}{t_{wc}}$$ $I_{13}=0.54$

Transverse stiffener checks

Maximum flange force that can be delivered to unstiffened column flange

$$R_{n.fy}:=\frac{F_{yc}\cdot Y_c\cdot t_{fc}^{\;2}}{d_{xb}-t_{fb}}$$ $R_{n.fy}=484.031\ \textit{kip}$

Required strength of transverse stiffeners

$$R_{u.st}:=P_{uf}-min\left(0.9\cdot R_{n.fy},1.0\cdot R_{n.wy},0.75\cdot R_{n.wc}\right)$$ $R_{u.st}=103.713\ \textit{kip}$

Transverse stiffener yielding check

Nominal strength of striffener in yielding

$$R_{n.sty} := 2 \cdot F_{yp} \cdot t_{st} \cdot \left(B_{st} - k_{st}\right) \qquad R_{n.sty} = 216\ \textit{kip}$$

Interaction ratio for stiffener yielding

$$I_{14} := \frac{R_{u.st}}{0.9 \cdot R_{n.sty}} \qquad I_{14} = 0.534$$

Transverse stiffener shear check

Nominal strength of stiffener in shear

$$R_{n.stv} := 0.6 \cdot F_{yp} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot t_{st} \qquad R_{n.stv} = 281.448\ \textit{kip}$$

Interaction ratio for stiffener in shear

$$I_{15} := \frac{R_{u.st}}{2 \cdot 0.9 \cdot R_{n.stv}} \qquad I_{15} = 0.205$$

Transverse stiffener thickness check

Minimum thickness of stiffener

$$t_{st.min} := \max\left(\frac{\frac{B_{st}}{\textit{in}} \cdot \sqrt{\frac{F_{yp}}{\textit{ksi}}}}{95}\ \textit{in}, \frac{t_{fb}}{2}\right) \qquad t_{st.min} = 0.253\ \textit{in}$$

Interaction ratio for stiffener thickness

$$I_{16} := \frac{t_{st.min}}{t_{st}} \qquad I_{16} = 0.253$$

Transverse stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w4} := 1.5 \cdot 0.6\ F_{EXX} \cdot \sqrt{2} \cdot t_{w4} \qquad R_{n.w4} = 44.548\ \frac{\textit{kip}}{\textit{in}}$$

Interaction ratio for weld

$$I_{17} := \frac{0.9 \cdot F_{yp} \cdot t_{st}}{0.75 \cdot R_{n.w4}} \qquad I_{17} = 0.97$$

Transverse stiffener to web weld check

Nominal strength of stiffener to web weld

$$R_{n.w5} := 2 \cdot 0.6\ F_{EXX} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot \sqrt{2} \cdot t_{w5} \qquad R_{n.w5} = 386.971\ \textit{kip}$$

Interaction ratio for stiffener to web weld

$$I_{18} := \frac{R_{u.st}}{0.75 \cdot R_{n.w5}} \qquad I_{18} = 0.357$$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

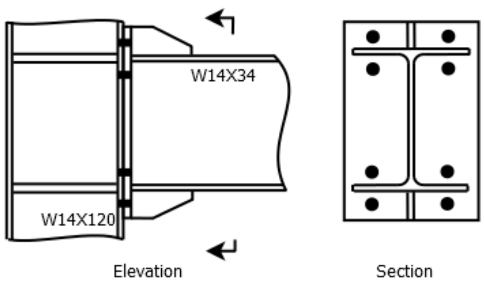
Table 2: Validation problem 1 results

	Interaction Ratio		
Check	Calculated	Osoconn	Result
Bolt tension check	0.917	0.917	OK
Bolt shear check	0.217	0.217	OK
Bolt bearing on end plate check	0.084	0.084	OK
Bolt bearing at column flange	0.117	0.117	OK
Plate thickness check	0.972	0.972	OK
Plate shear yielding check	0.439	0.439	OK
Plate shear rupture check	0.453	0.453	OK
Beam web to plate weld tension check	0.582	0.582	OK
Beam web to plate weld shear check	0.644	0.644	OK
Column flange flexural yielding	0.83	0.83	OK
Column web yielding strength	0.0	0.0	OK
Column web crippling check	0.0	0.0	OK
Column panel shear strength	0.964	0.964	OK
Web shear buckling check	0.54	0.54	OK
Stiffener yielding check	0.534	0.534	OK
Stiffener shear check	0.205	0.205	OK
Stiffener thickness check	0.253	0.253	OK
Stiffener to flange weld check	0.97	0.97	OK
Stiffener to web weld check	0.357	0.357	OK

2.3 Validation Problem 2

Problem Statement

Design a 4-bolt stiffened extended end-plate moment connection for a W14X34 beam framing into the flange of a W14X120 column using the LRFD method. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 45kip, and the axial force in the column is 350kip. The beam and column are grade ASTM A992. Plates are of grade ASTM A36. The bolts are ASTM A3125 A325 slip critical type.



Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 36\ ksi$
Tensile strength	$F_{up} := 58\ ksi$
Material grade of beam	ASTM A992
Yield strength	$F_{yb} := 50\ ksi$
Tensile strength	$F_{ub} := 65\ ksi$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.1$
Material grade of column	ASTM A992
Yield strength	$F_{yc} := 50\ ksi$
Tensile strength	$F_{uc} := 65\ ksi$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 70\ ksi$
Material specification for bolts	ASTM 3125 A325
Tensile strength	$F_{nt} := 90\ ksi$
Shear strength	$F_{nv} := 54\ ksi$
Young's modulus for steel	$E := 29000\ ksi$

Design Forces

Shear force in beam	$V_u := 45\ kip$
Axial force in column	$P_{uc} := 350\ kip$

Connection Geometry

Beam section W14X34

Section depth $d_{xb} := 14\ in$

Flange width $b_{fb} := 6.75\ in$

Flange thickness $t_{fb} := 0.455\ in$

Web thickness $t_{wb} := 0.285\ in$

Design distance from outer face to fillet edge $k_{des} := 0.855\ in$

Detailing distance from outer face to fillet edge $k_{det} := 1.1875\ in$

Plastic section modulus $Z_x := 54.6\ in^3$

Column section W14X120

Section depth $d_{xc} := 14.5\ in$

Flange width $b_{fc} := 14.7\ in$

Flange thickness $t_{fc} := 0.94\ in$

Web thickness $t_{wc} := 0.59\ in$

Cross section area of column $A_c := 35.3\ in^2$

Design dist form outer face to fillet edge $k_c := 1.54\ in$

End-plate width $B := 12\ in$

End-plate height $H := 23\ in$

End-plate thickness $t := 1\ in$

Bolt diameter $d_b := 1.125\ in$

Bolt hole diameter $d_{bh} := 1.188\ in$

Inner bolt pitch $p_i := 1.75\ in$

Outer bolt pitch $p_o := 2.25\ in$

Bolt gage $g := 5.5\ in$

Plate stiffener thickness $t_{sp} := 0.5\ in$

Plate stiffener length $l_{sp} := 8\ in$

Web weld thickness $w_1 := 0.25\ in$

Flange weld reinforcement $w_2 := 0.16\ in$

Plate stiffener weld $w_3 := 0.25\ in$

Column stiffener thickness $t_{st} := 0.75\ in$

Width of column stiffener $B_{st} := 4\ in$

Column stiffener clip dimension $k_{st} := 1\ in$

Thickness of stiffener to flange weld $t_{w4} := 0.5\ in$

Thickness of stiffener to web weld $t_{w5} := 0.25\ in$

Difference between top of beam and top of column $a := 3\ in$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 275.275\ kip \cdot ft$$

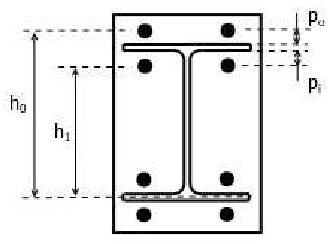
Design moment at face of column

$$M_u := M_p + V_u \cdot (l_{sp} + t) \qquad M_u = 309.025\ kip \cdot ft$$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}}$$ $P_{uf} = 273.776\ kip$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$$h_0 := d_{xb} - 0.5\ t_{fb} + p_o$$ $h_0 = 16.023\ in$

$$h_1 := d_{xb} - t_{fb} - p_i$$ $h_1 = 11.795\ in$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2 \cdot M_u}{\pi \cdot 0.75 \cdot F_{nt} \cdot \sum h}}$$ $d_{b.min} = 1.121\ in$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b}$$ $I_0 = 0.997$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot {d_b}^2}{4}$$ $A_b = 0.994\ in^2$

Nominal strength of bolt is shear

$$R_{n.bv} := F_{nv} \cdot A_b$$ $R_{n.bv} = 53.677\ kip$

Interaction ratio in bolt shear

$$I_1 := \frac{V_u}{4 \cdot 0.75 \cdot R_{n.bv}}$$ $I_1 = 0.279$

Bolt bearing on end plate check

Bolt hole edge to edge distance

$$ed_1 := 0.5 \cdot \left(H - d_{xb}\right) - p_o - 0.5 \cdot d_{bh}$$ $ed_1 = 1.656\ in$

Bolt hole to hole edge distance

$$s_1 := p_i + p_o + t_{fb} - d_{bh}$$ $s_1 = 3.267\ in$

Clear distances for bearing calculation

For outer bolt

$$L_{co} := min\left(ed_1, s_1\right) \qquad L_{co} = 1.656\ in$$

For inner bolt

$$L_{ci} := s_1 \qquad L_{ci} = 3.267\ in$$

Nominal strength in bearing

For outer bolts

$$R_{n.bbpo} := 2 \cdot min\left(1.2 \cdot L_{co} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbpo} = 230.515\ kip$$

For inner bolts

$$R_{n.bbpi} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbpi} = 313.2\ kip$$

Net bearing strength

$$R_{n.bbp} := R_{n.bbpi} + R_{n.bbpo} \qquad R_{n.bbp} = 543.715\ kip$$

Interaction ratio in bearing at end plate

$$I_2 := \frac{V_u}{0.75 \cdot R_{n.bbp}} \qquad I_2 = 0.11$$

Bolt bearing at column flange

Bolt hole edge to edge of column

$$ed_2 := a - p_o - 0.5\ d_{bh} \qquad ed_2 = 0.156\ in$$

Clear distances for bearing calculation

For outer bolt

$$L_{co2} := min\left(ed_2, s_1\right) \qquad L_{co} = 1.656\ in$$

Nominal strength in bearing

For outer bolts

$$R_{n.bbco} := 2 \cdot min\left(1.2 \cdot L_{co2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbco} = 22.876\ kip$$

For inner bolts

$$R_{n.bbci} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbci} = 329.94\ kip$$

Net bearing strength

$$R_{n.bbc} := R_{n.bbci} + R_{n.bbco} \qquad R_{n.bbc} = 352.816\ kip$$

Interaction ratio in bearing at end plate

$$I_3 := \frac{V_u}{0.75 \cdot R_{n.bbc}} \qquad I_3 = 0.17$$

Plate thickness check

Bolt tension strength

$$P_t := F_{nt} \cdot A_b \qquad P_t = 89.462\ kip$$

No-prying moment strength of plate

$$M_{np} := 2 \cdot P_t \cdot \sum h \qquad M_{np} = 414.767\ kip \cdot ft$$

Dimension

$$s := \frac{1}{2} \cdot \sqrt{B \cdot g}$$ $s = 4.062\ in$

$$p'_i := min(p_i, s)$$ $p'_i = 1.75\ in$

Vertical edge distance of outer bolt

$$d_e := \frac{H - d_{xb} - 2\ p_o}{2}$$ $d_e = 2.25\ in$

End plate yield line parameter

$$Y_{p1} := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{p_i} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_o} + \frac{1}{2\ s} \right) \right) + \frac{2}{g} \left(h_1 \cdot (p_i + s) + h_0 \cdot (d_e + p_o) \right)$$

$$Y_{p2} := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{p_i} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_o} + \frac{1}{s} \right) \right) + \frac{2}{g} \left(h_1 \cdot (p_i + s) + h_0 \cdot (s + p_o) \right)$$

$$Y_p := \mathbf{if}\left(d_e < s, Y_{p1}, Y_{p2}\right)$$ $Y_p = 163.569\ in$

Required end plate thickness

$$t_{p.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yp} \cdot Y_p}}$$ $t_{p.min} = 0.884\ in$

Interaction ratio for plate thickness

$$I_4 := \frac{t_{p.min}}{t}$$ $I_4 = 0.884$

Beam web to plate weld tension check

Required strength of weld in tension

$$f_{wt} := F_{yb} \cdot t_{wb}$$ $f_{wt} = 14.25\ \frac{kip}{in}$

Nominal strength of weld in tension

$$f_{n.wt} := 2 \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}} \cdot 1.5$$ $f_{n.wt} = 22.274\ \frac{kip}{in}$

Interaction ratio for weld in tension

$$I_5 := \frac{f_{wt}}{0.75 \cdot f_{n.wt}}$$ $I_5 = 0.853$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws} := d_{xb} - t_{fb} - p_i - 6\ in - k_{des}$$ $l_{ws} = 4.94\ in$

Nominal strenght of weld in shear

$$V_{n.ws} := 2 \cdot l_{ws} \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}}$$ $V_{n.ws} = 73.355\ kip$

Interaction ratio for weld in shear

$$I_6 := \frac{V_u}{0.75 \cdot V_{n.ws}} \qquad I_6 = 0.818$$

End plate stiffener thickness check

Height of stiffener

$$h_{sp} := \frac{H - d_{xb}}{2} \qquad h_{sp} = 4.5\ in$$

Minimum thickness for stiffener buckling

$$t_{sp.buc} := 1.79 \cdot h_{sp} \cdot \sqrt{\frac{F_{yp}}{E}} \qquad t_{sp.buc} = 0.284\ in$$

Minimum required thickness for stiffener

$$t_{sp.min} := \max\left(t_{sp.buc}, t_{wb} \cdot \left(\frac{F_{yb}}{F_{yp}}\right)\right) \qquad t_{sp.min} = 0.396\ in$$

Interaction ratio for stiffener thickness

$$I_7 := \frac{t_{sp.min}}{t_{sp}} \qquad I_7 = 0.792$$

End plate stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w3} := 0.6\ F_{EXX} \cdot \sqrt{2} \cdot w_3 \qquad R_{n.w3} = 14.849\ \frac{kip}{in}$$

Interaction ratio in stiffener buckling

$$I_8 := \frac{0.6 \cdot F_{yp} \cdot t_{sp}}{0.75 \cdot R_{n.w3}} \qquad I_8 = 0.97$$

Column flange flexural yielding

Dimensions

$$s_c := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} \qquad s_c = 4.496\ in$$

$$p_{si} := p_i + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{si} = 1.603\ in$$

$$p_{so} := p_o + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{so} = 2.103\ in$$

Yield line parameter for column flange

$$Y_c := \frac{b_{fc}}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s} + \frac{1}{p_{si}}\right) + h_0 \cdot \left(\frac{1}{s} + \frac{1}{p_{so}}\right)\right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(s + p_{si}\right) + h_0 \cdot \left(s + p_{so}\right)\right)$$

$$Y_c = 220.657\ in$$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yc} \cdot Y_c}} \qquad t_{fc.min} = 0.646\ in$$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}} \qquad I_9 = 0.687$$

Column web yielding strength

Web yielding factor

$$C_t := \text{if}\left(a < d_{xc}, 0.5, 1\right) \qquad C_t = 0.5$$

Bearing length of beam flange

$$N := t_{fb} + 2 \cdot w_2 \qquad N = 0.775 \ in$$

Nominal strength in web yielding

$$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t\right) + N\right) \cdot F_{yc} \cdot t_{wc} \qquad R_{n.wy} = 188.653 \ kip$$

Interaction ratio in web yielding

$$I_{10} := \frac{P_{uf}}{1.0 \cdot R_{n.wy}} \qquad I_{10} := 0$$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + \left(\frac{4 \cdot N}{d_{xc}} - 0.2\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc} := \text{if}\left(\left(a + 0.5 \cdot t_{fb}\right) \ge 0.5 \cdot d_{xc}, R_{n.wc1}, \text{if}\left(\left(N \div d_{xc}\right) < 0.2, R_{n.wc2}, R_{n.wc3}\right)\right)$$

$$R_{n.wc} = 228.509 \ kip$$

Interaction ratio for web crippling

$$I_{11} := \frac{P_{uf}}{0.75 \cdot R_{n.wc}} \qquad I_{11} := 0$$

Column panel shear strength

Axial yield strength of column

$$P_y := F_{yc} \cdot A_c \qquad P_y = 1765 \ kip$$

Panel zone shear strength

$$R_{n.pz1} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right)$$

$$R_{n.pz2} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_{uc}}{P_y}\right)$$

$$R_{n.pz} := \text{if}\left(P_{uc} \le 0.75 \cdot P_y, R_{n.pz1}, R_{n.pz2}\right) \qquad R_{n.pz} = 340.15\ kip$$

Interaction ratio in panel zone shear

$$I_{12} := \frac{P_{uf}}{0.9 \cdot R_{n.pz}} \qquad I_{12} = 0.894$$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min} := \frac{d_{xb} - t_{fb} + d_{xc} - 2 \cdot t_{fc}}{90} \qquad t_{wc.min} = 0.291\ in$$

Interaction ratio in web shear buckling

$$I_{13} := \frac{t_{wc.min}}{t_{wc}} \qquad I_{13} = 0.493$$

Transverse stiffener checks

Maximum flange force that can be delivered to unstiffened column flange

$$R_{n.fy} := \frac{F_{yc} \cdot Y_c \cdot {t_{fc}}^2}{d_{xb} - t_{fb}} \qquad R_{n.fy} = 719.723\ kip$$

Required strength of transverse stiffeners

$$R_{u.st} := P_{uf} - min\left(0.9 \cdot R_{n.fy}, 1.0 \cdot R_{n.wy}, 0.75 \cdot R_{n.wc}\right) \qquad R_{u.st} = 102.395\ kip$$

Transverse stiffener yielding check

Nominal strength of striffener in yielding

$$R_{n.sty} := 2 \cdot F_{yp} \cdot t_{st} \cdot \left(B_{st} - k_{st}\right) \qquad R_{n.sty} = 162\ kip$$

Interaction ratio for stiffener yielding

$$I_{14} := \frac{R_{u.st}}{0.9 \cdot R_{n.sty}} \qquad I_{14} = 0.702$$

Transverse stiffener shear check

Nominal strength of stiffener in shear

$$R_{n.stv} := 0.6 \cdot F_{yp} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot t_{st} \qquad R_{n.stv} = 172.044\ kip$$

Interaction ratio for stiffener in shear

$$I_{15} := \frac{R_{u.st}}{2 \cdot 0.9 \cdot R_{n.stv}} \qquad I_{15} = 0.331$$

Transverse stiffener thickness check

Minimum thickness of stiffener

$$t_{st.min} := \max\left(\frac{\frac{B_{st}}{in} \cdot \sqrt{\frac{F_{yp}}{ksi}}}{95}\ in, \frac{t_{fb}}{2}\right) \qquad t_{st.min} = 0.253\ in$$

Interaction ratio for stiffener thickness

$$I_{16} := \frac{t_{st.min}}{t_{st}} \qquad I_{16} = 0.337$$

Transverse stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w4} := 1.5 \cdot 0.6\ F_{EXX} \cdot \sqrt{2} \cdot t_{w4} \qquad R_{n.w4} = 44.548\ \frac{kip}{in}$$

Interaction ratio for weld

$$I_{17} := \frac{0.9 \cdot F_{yp} \cdot t_{st}}{0.75 \cdot R_{n.w4}} \qquad I_{17} = 0.727$$

Transverse stiffener to web weld check

Nominal strength of stiffener to web weld

$$R_{n.w5} := 2 \cdot 0.6\ F_{EXX} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot \sqrt{2} \cdot t_{w5} \qquad R_{n.w5} = 315.398\ kip$$

Interaction ratio for stiffener to web weld

$$I_{18} := \frac{R_{u.st}}{0.75 \cdot R_{n.w5}} \qquad I_{18} = 0.433$$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

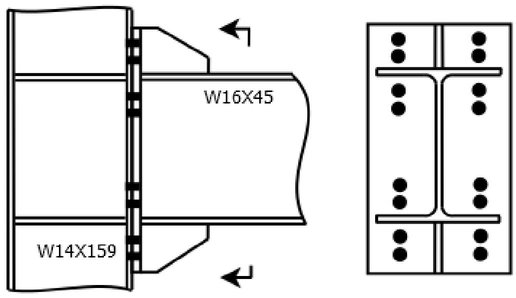
Table 3: Validation problem 2 results

	Interaction Ratio		
Check	Calculated	Osoconn	Result
Bolt tension check	0.997	0.997	OK
Bolt shear rupture check	0.279	0.279	OK
Bolt bearing on end plate check	0.11	0.11	OK
Bolt bearing at column flange	0.17	0.17	OK
Plate thickness check	0.884	0.884	OK
Beam web to plate weld tension check	0.853	0.853	OK
Beam web to plate weld shear check	0.818	0.818	OK
End plate stiffener thickness check	0.792	0.792	OK
End plate stiffener to flange weld check	0.97	0.97	OK
Column flange flexural yielding	0.687	0.688	OK
Column web yielding strength	0.0	0.0	OK
Column web crippling check	0.0	0.0	OK
Column panel shear strength	0.894	0.894	OK
Web shear buckling check	0.493	0.493	OK
Transverse stiffener yielding check	0.702	0.702	OK
Transverse stiffener shear check	0.331	0.331	OK
Transverse stiffener thickness check	0.337	0.337	OK
Transverse stiffener to flange weld check	0.727	0.727	OK
Transverse stiffener to web weld check	0.433	0.433	OK

2.4 Validation Problem 3

Problem Statement

Design a 4-bolt stiffened extended end-plate moment connection for a W16X45 beam framing into the flange of a W14X159 column using the LRFD method. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 60kip, and the axial force in the column is 590kip. The beam and column are grade ASTM A992. Plates are of grade ASTM A36. The bolts are ASTM A3125 A490 slip critical type.



Elevation　　　　Section

Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 36\ ksi$
Tensile strength	$F_{up} := 58\ ksi$
Material grade of beam	ASTM A992
Yield strength	$F_{yb} := 50\ ksi$
Tensile strength	$F_{ub} := 65\ ksi$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.1$
Material grade of column	ASTM A992
Yield strength	$F_{yc} := 50\ ksi$
Tensile strength	$F_{uc} := 65\ ksi$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 70\ ksi$
Material specification for bolts	ASTM 3125 A490
Tensile strength	$F_{nt} := 113\ ksi$
Shear strength	$F_{nv} := 68\ ksi$
Young's modulus for steel	$E := 29000\ ksi$

Design Forces

Shear force in beam	$V_u := 60\ kip$
Axial force in column	$P_{uc} := 590\ kip$

Connection Geometry

Beam section	W16X45
Section depth	$d_{xb} := 16.1\ in$
Flange width	$b_{fb} := 7.04\ in$
Flange thickness	$t_{fb} := 0.565\ in$
Web thickness	$t_{wb} := 0.345\ in$
Design distance from outer face to fillet edge	$k_{des} := 0.967\ in$
Detailing distance from outer face to fillet edge	$k_{det} := 1.25\ in$
Plastic section modulus	$Z_x := 82.3\ in^3$
Column section	W14X159
Section depth	$d_{xc} := 15\ in$
Flange width	$b_{fc} := 15.6\ in$
Flange thickness	$t_{fc} := 1.19\ in$
Web thickness	$t_{wc} := 0.745\ in$
Cross section area of column	$A_c := 46.7\ in^2$
Design dist form outer face to fillet edge	$k_c := 1.79\ in$
End-plate width	$B := 12\ in$
End-plate height	$H := 30\ in$
End-plate thickness	$t := 1.25\ in$
Bolt diameter	$d_b := 1.125\ in$
Bolt hole diameter	$d_{bh} := 1.188\ in$
Inner bolt pitch	$p_i := 1.75\ in$
Outer bolt pitch	$p_o := 1.75\ in$
Bolt to bolt pitch	$p_b := 3.5\ in$
Bolt gage	$g := 6\ in$
Plate stiffener thickness	$t_{sp} := 0.5\ in$
Plate stiffener length	$l_{sp} := 13\ in$
Web weld thickness	$w_1 := 0.313\ in$
Flange weld reinforcement	$w_2 := 0.16\ in$
Plate stiffener weld	$w_3 := 0.25\ in$
Column stiffener thickness	$t_{st} := 0.75\ in$
Width of column stiffener	$B_{st} := 6\ in$
Column stiffener clip dimension	$k_{st} := 1\ in$
Thickness of stiffener to flange weld	$t_{w4} := 0.5\ in$
Thickness of stiffener to web weld	$t_{w5} := 0.25\ in$
Difference between top of beam and top of column	$a := 12\ in$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 414.929\ kip \cdot ft$$

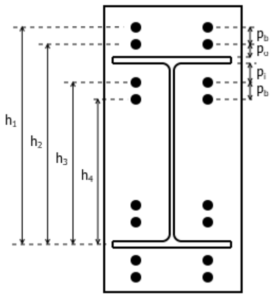
Design moment at face of column

$$M_u := M_p + V_u \cdot (l_{sp} + t)$$ $M_u = 486.179\ kip \cdot ft$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}}$$ $P_{uf} = 375.549\ kip$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$$h_1 := d_{xb} - 0.5\ t_{fb} + p_o + p_b$$ $h_1 = 21.068\ in$

$$h_2 := h_1 - p_b$$ $h_2 = 17.568\ in$

$$h_3 := d_{xb} - t_{fb} - p_i$$ $h_3 = 13.785\ in$

$$h_4 := h_3 - p_b$$ $h_4 = 10.285\ in$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2 \cdot M_u}{\pi \cdot 0.75 \cdot F_{nt} \cdot \sum h}}$$ $d_{b.min} = 0.836\ in$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b}$$ $I_0 = 0.743$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot d_b^2}{4}$$ $A_b = 0.994\ in^2$

Nominal strength of bolt is shear

$$R_{n.bv} := F_{nv} \cdot A_b$$ $R_{n.bv} = 67.593\ kip$

Interaction ratio in bolt shear

$$I_1 := \frac{V_u}{8 \cdot 0.75 \cdot R_{n.bv}}$$ $I_1 = 0.148$

Bolt bearing on end plate check

Vertical edge distance of outer bolt

$$d_e := \frac{H - d_{xb} - 2\,(p_o + p_b)}{2} \qquad d_e = 1.7\ in$$

Clear distances for bearing calculation for each bolt row

$$L_{c1} := min\left(d_e - 0.5\ d_{bh}, p_b - d_{bh}\right) \qquad L_{c1} = 1.106\ in$$

$$L_{c2} := min\left(p_b - d_{bh}, p_o + p_i + t_{fb} - d_{bh}\right) \qquad L_{c2} = 2.312\ in$$

$$L_{c3} := L_{c2} \qquad L_{c3} = 2.312\ in$$

$$L_{c4} := p_b - d_{bh} \qquad L_{c4} = 2.312\ in$$

Nominal strength in bearing for each bolt row

$$R_{n.bbp1} := 2 \cdot min\left(1.2 \cdot L_{c1} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbp1} = 192.444\ kip$$

$$R_{n.bbp2} := 2 \cdot min\left(1.2 \cdot L_{c2} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbp2} = 391.5\ kip$$

$$R_{n.bbp3} := 2 \cdot min\left(1.2 \cdot L_{c3} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbp3} = 391.5\ kip$$

$$R_{n.bbp4} := 2 \cdot min\left(1.2 \cdot L_{c4} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right) \qquad R_{n.bbp4} = 391.5\ kip$$

Net bearing strength

$$R_{n.bbp} := R_{n.bbp1} + R_{n.bbp2} + R_{n.bbp3} + R_{n.bbp4} \qquad R_{n.bbp} = \left(1.367 \cdot 10^3\right)\ kip$$

Interaction ratio in bearing at end plate

$$I_2 := \frac{V_u}{0.75 \cdot R_{n.bbp}} \qquad I_2 = 0.059$$

Bolt bearing at column flange

Clear distances for bearing calculation of outer bolt

$$L_{c1} := min\left(a - p_o - p_b - 0.5\ d_{bh}, p_b - d_{bh}\right) \qquad L_{c1} = 2.312\ in$$

Nominal strength in bearing for each bolt row

$$R_{n.bbc1} := 2 \cdot min\left(1.2 \cdot L_{c1} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbc1} = 417.69\ kip$$

$$R_{n.bbc2} := 2 \cdot min\left(1.2 \cdot L_{c2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbc1} = 417.69\ kip$$

$$R_{n.bbc3} := 2 \cdot min\left(1.2 \cdot L_{c3} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbc1} = 417.69\ kip$$

$$R_{n.bbc4} := 2 \cdot min\left(1.2 \cdot L_{c4} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right) \qquad R_{n.bbc1} = 417.69\ kip$$

Net bearing strength

$$R_{n.bbc} := R_{n.bbc1} + R_{n.bbc2} + R_{n.bbc3} + R_{n.bbc4} \qquad R_{n.bbc} = \left(1.671 \cdot 10^3\right)\ kip$$

Interaction ratio in bearing at end plate

$$I_3 := \frac{V_u}{0.75 \cdot R_{n.bbc}} \qquad I_3 = 0.048$$

Plate thickness check
Bolt tension strength

$$P_t := F_{nt} \cdot A_b \qquad P_t = 112.324\ kip$$

No-prying moment strength of plate

$$M_{np} := 2 \cdot P_t \cdot \sum h \qquad M_{np} = 14086.579\ kip \cdot in$$

Dimension

$$s := \frac{1}{2} \cdot \sqrt{B \cdot g} \qquad s = 4.243\ in$$

$$p'_i := min\left(p_i, s\right) \qquad p'_i = 1.75\ in$$

End plate yield line parameter

$$Y_{p1} := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{2\ d_e}\right) + h_2 \cdot \left(\frac{1}{p_o}\right) + h_3 \cdot \left(\frac{1}{p_i}\right) + h_4 \cdot \left(\frac{1}{s}\right)\right)$$

$$Y_{p1} := Y_{p1} + \frac{2}{g}\left(h_1 \cdot \left(d_e + \frac{p_b}{4}\right) + h_2 \cdot \left(p_o + \frac{3\ p_b}{4}\right) + h_3 \cdot \left(p_i + \frac{p_b}{4}\right) + h_4 \cdot \left(s + \frac{3\ p_b}{4}\right) + {p_b}^2\right) + g$$

$$Y_{p2} := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s}\right) + h_2 \cdot \left(\frac{1}{p_o}\right) + h_3 \cdot \left(\frac{1}{p_i}\right) + h_4 \cdot \left(\frac{1}{s}\right)\right)$$

$$Y_{p2} := Y_{p2} + \frac{2}{g}\left(h_1 \cdot \left(s + \frac{p_b}{4}\right) + h_2 \cdot \left(p_o + \frac{3\ p_b}{4}\right) + h_3 \cdot \left(p_i + \frac{p_b}{4}\right) + h_4 \cdot \left(s + \frac{3\ p_b}{4}\right) + {p_b}^2\right) + g$$

$$Y_p := \text{if}\left(d_e < s, Y_{p1}, Y_{p2}\right) \qquad Y_p = 248.609\ in$$

Required end plate thickness

$$t_{p.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yp} \cdot Y_p}} \qquad t_{p.min} = 1.207\ in$$

Interaction ratio for plate thickness

$$I_4 := \frac{t_{p.min}}{t} \qquad I_4 = 0.965$$

Beam web to plate weld tension check
Required strength of weld in tension

$$f_{wt} := F_{yb} \cdot t_{wb} \qquad f_{wt} = 17.25\ \frac{kip}{in}$$

Nominal strength of weld in tension

$$f_{n.wt} := 2 \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}} \cdot 1.5 \qquad f_{n.wt} = 27.887\ \frac{kip}{in}$$

Interaction ratio for weld in tension

$$I_5 := \frac{f_{wt}}{0.75 \cdot f_{n.wt}} \qquad I_5 = 0.825$$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws}:=d_{xb}-t_{fb}-p_i-6\ in-k_{des} \qquad l_{ws}=6.818\ in$$

Nominal strenght of weld in shear

$$V_{n.ws}:=2\cdot l_{ws}\cdot 0.6\cdot F_{EXX}\cdot\frac{w_1}{\sqrt{2}} \qquad V_{n.ws}=126.755\ kip$$

Interaction ratio for weld in shear

$$I_6:=\frac{V_u}{0.75\cdot V_{n.ws}} \qquad I_6=0.631$$

Plate stiffener thickness check

Height of stiffener

$$h_{sp}:=\frac{H-d_{xb}}{2} \qquad h_{sp}=6.95\ in$$

Minimum thickness for stiffener buckling

$$t_{sp.buc}:=1.79\cdot h_{sp}\cdot\sqrt{\frac{F_{yp}}{E}} \qquad t_{sp.buc}=0.438\ in$$

Minimum required thickness for stiffener

$$t_{sp.min}:=\max\left(t_{sp.buc},t_{wb}\cdot\left(\frac{F_{yb}}{F_{yp}}\right)\right) \qquad t_{sp.min}=0.479\ in$$

Interaction ratio for stiffener thickness

$$I_7:=\frac{t_{sp.min}}{t_{sp}} \qquad I_7=0.958$$

Plate stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w3}:=0.6\ F_{EXX}\cdot\sqrt{2}\cdot w_3 \qquad R_{n.w3}=14.849\ \frac{kip}{in}$$

Interaction ratio in stiffener buckling

$$I_8:=\frac{0.6\cdot F_{yp}\cdot t_{sp}}{0.75\cdot R_{n.w3}} \qquad I_8=0.97$$

Column flange flexural yielding

Dimensions

$$s_c:=\frac{1}{2}\cdot\sqrt{b_{fc}\cdot g} \qquad s_c=4.837\ in$$

$$p_{si}:=p_i+0.5\cdot t_{fb}-0.5\cdot t_{st} \qquad p_{si}=1.658\ in$$

$$p_{so}:=p_o+0.5\cdot t_{fb}-0.5\cdot t_{st} \qquad p_{so}=1.658\ in$$

Yield line parameter for column flange

$$Y_c:=\frac{b_{fc}}{2}\cdot\left(h_1\cdot\left(\frac{1}{s}\right)+h_2\cdot\left(\frac{1}{p_{so}}\right)+h_3\cdot\left(\frac{1}{p_{si}}\right)+h_4\cdot\left(\frac{1}{s}\right)\right)$$

$$Y_c := Y_c + \frac{2}{g} \cdot \left(h_1 \cdot \left(s + \frac{p_b}{4} \right) + h_2 \cdot \left(p_{so} + \frac{3\ p_b}{4} \right) + h_3 \cdot \left(p_{si} + \frac{p_b}{4} \right) + h_4 \cdot \left(s + \frac{3\ p_b}{4} \right) + {p_b}^2 \right) + g$$

$Y_c = 311.463\ in$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yc} \cdot Y_c}}$$ $t_{fc.min} = 0.915\ in$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}}$$ $I_9 = 0.769$

Column web yielding strength

Web yielding factor

$$C_t := \text{if} \left(a < d_{xc}, 0.5, 1 \right)$$ $C_t = 0.5$

Bearing length of beam flange

$$N := t_{fb} + 2 \cdot w_2$$ $N = 0.885\ in$

Nominal strength in web yielding

$$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t \right) + N \right) \cdot F_{yc} \cdot t_{wc}$$ $R_{n.wy} = 279.561\ kip$

Interaction ratio in web yielding

$$I_{10} := \frac{P_{uf}}{1.0 \cdot R_{n.wy}}$$ $I_{10} := 0$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot {t_{wc}}^2 \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}} \right) \cdot \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2} := 0.4 \cdot {t_{wc}}^2 \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}} \right) \cdot \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3} := 0.4 \cdot {t_{wc}}^2 \cdot \left(1 + \left(\frac{4 \cdot N}{d_{xc}} - 0.2 \right) \cdot \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc} := \text{if} \left(\left(a + 0.5 \cdot t_{fb} \right) \ge 0.5 \cdot d_{xc}, R_{n.wc1}, \text{if} \left(\left(N \div d_{xc} \right) < 0.2, R_{n.wc2}, R_{n.wc3} \right) \right)$$

$R_{n.wc} = 734.991\ kip$

Interaction ratio for web crippling

$$I_{11} := \frac{P_{uf}}{0.75 \cdot R_{n.wc}}$$ $I_{11} := 0$

Column panel shear strength

Axial yield strength of column

$$P_y := F_{yc} \cdot A_c$$ $P_y = 2335\ kip$

Panel zone shear strength

$$R_{n.pz1} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right)$$

$$R_{n.pz2} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_{uc}}{P_y}\right)$$

$$R_{n.pz} := \text{if}\left(P_{uc} \le 0.75 \cdot P_y , R_{n.pz1} , R_{n.pz2}\right) \qquad R_{n.pz} = 458.741\ \textit{kip}$$

Interaction ratio in panel zone shear

$$I_{12} := \frac{P_{uf}}{0.9 \cdot R_{n.pz}} \qquad I_{12} = 0.91$$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min} := \frac{d_{xb} - t_{fb} + d_{xc} - 2 \cdot t_{fc}}{90} \qquad t_{wc.min} = 0.313\ \textit{in}$$

Interaction ratio in web shear buckling

$$I_{13} := \frac{t_{wc.min}}{t_{wc}} \qquad I_{13} = 0.42$$

Transverse stiffener checks

Inner bolt to bolt distance

$$c := p_o + p_i + t_{fb} \qquad c = 4.065\ \textit{in}$$

Yield line parameter for unstiffened column flange

$$Y`_c := \frac{b_{fc}}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s}\right) + h_4 \cdot \left(\frac{1}{s}\right)\right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(p_b + \frac{c}{2} + s\right) + h_2 \cdot \left(\frac{p_b}{2} + \frac{c}{4}\right) + h_3 \cdot \left(\frac{p_b}{2} + \frac{c}{2}\right) + h_4 \cdot (s)\right) + \frac{g}{2}$$

$$Y`_c = 177.411\ \textit{in}$$

Maximum flange force that can be delivered to unstiffened column flange

$$R_{n.fy} := \frac{F_{yc} \cdot Y`_c \cdot t_{fc}^{\ 2}}{d_{xb} - t_{fb}} \qquad R_{n.fy} = 808.6\ \textit{kip}$$

Required strength of transverse stiffeners

$$R_{u.st} := P_{uf} - min\left(0.9 \cdot R_{n.fy} , 1.0 \cdot R_{n.wy} , 0.75 \cdot R_{n.wc}\right) \qquad R_{u.st} = 95.988\ \textit{kip}$$

Stiffener yielding check

Nominal strength of striffener in yielding

$$R_{n.sty} := 2 \cdot F_{yp} \cdot t_{st} \cdot \left(B_{st} - k_{st}\right) \qquad R_{n.sty} = 270\ \textit{kip}$$

Interaction ratio for stiffener yielding

$$I_{14} := \frac{R_{u.st}}{0.9 \cdot R_{n.sty}} \qquad I_{14} = 0.395$$

Stiffener shear check

Nominal strength of stiffener in shear

$$R_{n.stv} := 0.6 \cdot F_{yp} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot t_{st} \qquad R_{n.stv} = 172.044 \ \textit{kip}$$

Interaction ratio for stiffener in shear

$$I_{15} := \frac{R_{u.st}}{2 \cdot 0.9 \cdot R_{n.stv}} \qquad I_{15} = 0.31$$

Stiffener thickness check

Minimum thickness of stiffener

$$t_{st.min} := \max\left(\frac{\frac{B_{st}}{\textit{in}} \cdot \sqrt{\frac{F_{yp}}{\textit{ksi}}}}{95} \ \textit{in}, \frac{t_{fb}}{2}\right) \qquad t_{st.min} = 0.379 \ \textit{in}$$

Interaction ratio for stiffener thickness

$$I_{16} := \frac{t_{st.min}}{t_{st}} \qquad I_{16} = 0.505$$

Stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w4} := 1.5 \cdot 0.6 \ F_{EXX} \cdot \sqrt{2} \cdot t_{w4} \qquad R_{n.w4} = 44.548 \ \frac{\textit{kip}}{\textit{in}}$$

Interaction ratio for weld

$$I_{17} := \frac{0.9 \cdot F_{yp} \cdot t_{st}}{0.75 \cdot R_{n.w4}} \qquad I_{17} = 0.727$$

Stiffener to web weld check

Nominal strength of stiffener to web weld

$$R_{n.w5} := 2 \cdot 0.6 \ F_{EXX} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot \sqrt{2} \cdot t_{w5} \qquad R_{n.w5} = 315.398 \ \textit{kip}$$

Interaction ratio for stiffener to web weld

$$I_{18} := \frac{R_{u.st}}{0.75 \cdot R_{n.w5}} \qquad I_{18} = 0.406$$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

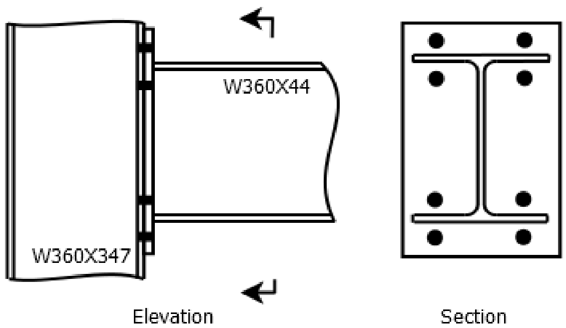
Table 4: Validation problem 3 results

Check	Interaction Ratio		Result
	Calculated	Osoconn	
Bolt tension check	0.743	0.743	OK
Bolt shear check	0.148	0.148	OK
Bolt bearing on end plate check	0.059	0.059	OK
Bolt bearing at column flange	0.048	0.048	OK
Plate thickness check	0.965	0.965	OK
Beam web to plate weld tension check	0.825	0.825	OK
Beam web to plate weld shear check	0.631	0.631	OK
End plate stiffener thickness check	0.958	0.958	OK
End plate stiffener to flange weld check	0.97	0.97	OK
Column flange flexural yielding	0.769	0.77	OK
Column web yielding strength	0.0	0.0	OK
Column web crippling check	0.0	0.0	OK
Column panel shear strength	0.91	0.91	OK
Web shear buckling check	0.42	0.42	OK
Stiffener yielding check	0.395	0.395	OK
Stiffener shear check	0.31	0.31	OK
Stiffener thickness check	0.505	0.505	OK
Stiffener to flange weld check	0.727	0.727	OK
Stiffener to web weld check	0.406	0.406	OK

2.5 Validation Problem 4

Problem Statement

Design a 4-bolt unstiffened extended end-plate moment connection for a W360X44 beam framing into the flange of a W360X347 column using the ASD method. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 36kN, and the axial force in the column is 455kN. The beam, column and plates are of grade ASTM A36. The bolts are ASTM A3125 A490.



Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 250\ MPa$
Tensile strength	$F_{up} := 400\ MPa$
Material grade of beam	ASTM A36
Yield strength	$F_{yb} := 250\ MPa$
Tensile strength	$F_{ub} := 400\ MPa$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.5$
Material grade of column	ASTM A36
Yield strength	$F_{yc} := 250\ MPa$
Tensile strength	$F_{uc} := 400\ MPa$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 482\ MPa$
Material specification for bolts	ASTM 3125 A490
Tensile strength	$F_{nt} := 780\ MPa$
Shear strength	$F_{nv} := 469\ MPa$
Young's modulus for steel	$E := 200000\ MPa$

Design Forces

Shear force in beam	$V_u := 36\ kN$
Axial force in column	$P_{uc} := 455\ kN$

Connection Geometry

Beam section — W360X44

Section depth — $d_{xb} := 351\ mm$

Flange width — $b_{fb} := 171\ mm$

Flange thickness — $t_{fb} := 9.78\ mm$

Web thickness — $t_{wb} := 6.86\ mm$

Design distance from outer face to fillet edge — $k_{des} := 19.9\ mm$

Detailing distance from outer face to fillet edge — $k_{det} := 28.6\ mm$

Plastic section modulus — $Z_x := 775\ cm^3$

Column section — W360X347

Section depth — $d_{xc} := 406\ mm$

Flange width — $b_{fc} := 404\ mm$

Flange thickness — $t_{fc} := 43.7\ mm$

Web thickness — $t_{wc} := 27.2\ mm$

Cross section area of column — $A_c := 44200\ mm^2$

Design dist form outer face to fillet edge — $k_c := 58.9\ mm$

End-plate width — $B := 400\ mm$

End-plate height — $H := 650\ mm$

End-plate thickness — $t := 30\ mm$

Bolt diameter — $d_b := 27\ mm$

Bolt hole diameter — $d_{bh} := 30\ mm$

Inner bolt pitch — $p_i := 40\ mm$

Outer bolt pitch — $p_o := 100\ mm$

Bolt gage — $g := 150\ mm$

Web weld thickness — $w_1 := 6\ mm$

Flange weld reinforcement — $w_2 := 4\ mm$

Difference between top of beam and top of column — $a := 100\ mm$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 319.688\ kN \cdot m$$

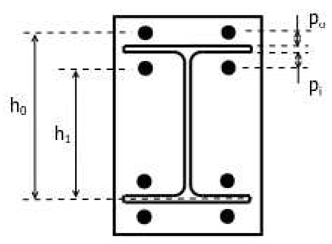
Design moment at face of column

$$M_u := M_p + V_u \cdot \left(min \left(\frac{d_{xb}}{2}, 3 \cdot b_{fb} \right) \right) \qquad M_u = 326.006\ kN \cdot m$$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}} \qquad P_{uf} = 955.411\ kN$$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$$h_0 := d_{xb} - 0.5\ t_{fb} + p_o \qquad h_0 = 446.11\ mm$$

$$h_1 := d_{xb} - t_{fb} - p_i \qquad h_1 = 301.22\ mm$$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2.0 \cdot 2 \cdot M_u}{\pi \cdot F_{nt} \cdot \sum h}} \qquad d_{b.min} = 26.685\ mm$$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b} \qquad I_0 = 0.988$$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot {d_b}^2}{4} \qquad A_b = 572.555\ mm^2$$

Nominal strength of bolt is shear

$$R_{n.bv} := F_{nv} \cdot A_b \qquad R_{n.bv} = 268.528\ kN$$

Interaction ratio in bolt shear

$$I_1 := \frac{2.0 \cdot V_u}{4 \cdot R_{n.bv}} \qquad I_1 = 0.067$$

Bolt bearing on end plate check

Bolt hole edge to edge distance

$$ed_1 := 0.5 \cdot \left(H - d_{xb}\right) - p_o - 0.5 \cdot d_{bh} \qquad ed_1 = 34.5\ mm$$

Bolt hole to hole edge distance

$$s_1 := p_i + p_o + t_{fb} - d_{bh} \qquad s_1 = 119.78\ mm$$

Clear distances for bearing calculation

For outer bolt

$$L_{co} := min\left(ed_1, s_1\right) \qquad L_{co} = 34.5\ mm$$

For inner bolt

$$L_{ci} := s_1 \qquad L_{ci} = 119.78\ mm$$

Nominal strength in bearing

For outer bolts

$R_{n.bbpo} := 2 \cdot min\left(1.2 \cdot L_{co} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbpo} = 993.6$ kN

For inner bolts

$R_{n.bbpi} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbpi} = 1555.2$ kN

Net bearing strength

$R_{n.bbp} := R_{n.bbpi} + R_{n.bbpo}$ $R_{n.bbp} = 2548.8$ kN

Interaction ratio in bearing at end plate

$I_2 := \dfrac{2.0 \cdot V_u}{R_{n.bbp}}$ $I_2 = 0.028$

Bolt bearing at column flange

Bolt hole edge to edge of column

$ed_2 := a - p_o - 0.5\ d_{bh}$ $ed_2 = -15$ mm

Clear distances for bearing calculation

For outer bolt

$L_{co2} := min\left(ed_2, s_1\right)$ $L_{co} = 34.5$ mm

Nominal strength in bearing

For outer bolts

$R_{n.bbco} := 2 \cdot min\left(1.2 \cdot L_{co2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbco} = -629.28$ kN

For inner bolts

$R_{n.bbci} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbci} = 2265.408$ kN

Net bearing strength

$R_{n.bbc} := R_{n.bbci} + R_{n.bbco}$ $R_{n.bbc} = 1636.128$ kN

Interaction ratio in bearing at end plate

$I_3 := \dfrac{2.0 \cdot V_u}{R_{n.bbc}}$ $I_3 = 0.044$

Plate thickness check

Bolt tension strength

$P_t := F_{nt} \cdot A_b$ $P_t = 446.593$ kN

No-prying moment strength of plate

$M_{np} := 2 \cdot P_t \cdot \sum h$ $M_{np} = 667.505$ $kN \cdot m$

Dimension

$s := \dfrac{1}{2} \cdot \sqrt{B \cdot g}$ $s = 122.474$ mm

$p'_i := min\left(p_i, s\right)$ $p'_i = 40$ mm

End plate yield line parameter

$$Y_p := \frac{B}{2} \cdot \left(h_1 \cdot \left(\frac{1}{p'_i} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_o} \right) - \frac{1}{2} \right) + \frac{2}{g} \left(h_1 \cdot (p'_i + s) \right)$$

$Y_p = 3442.751\ mm$

Required end plate thickness

$$t_{p.min} := \sqrt{\frac{1.11 \cdot 1.67 \cdot M_{np}}{2.0 \cdot F_{yp} \cdot Y_p}}$$ $t_{p.min} = 26.811\ mm$

Interaction ratio for plate thickness

$$I_4 := \frac{t_{p.min}}{t}$$ $I_4 = 0.894$

Plate shear yielding check

Nominal strength of plate in shear yielding

$$R_{n.py} := 0.6 \cdot F_{yp} \cdot B \cdot t$$ $R_{n.py} = 1800\ kN$

Interaction ratio in shear yielding

$$I_5 := \frac{1.5 \cdot P_{uf}}{2 \cdot R_{n.py}}$$ $I_5 = 0.398$

Plate shear rupture check

Net area of plate in shear

$$A_n := \left(B - 2 \cdot d_{bh}\right) \cdot t$$ $A_n = 102\ cm^2$

Nominal strength of plate in shear rupture

$$R_{n.pr} := 0.6 \cdot F_{up} \cdot A_n$$ $R_{n.pr} = 2448\ kN$

Interaction ratio in shear rupture

$$I_6 := \frac{2.0 \cdot P_{uf}}{2 \cdot R_{n.pr}}$$ $I_6 = 0.39$

Beam web to plate weld tension check

Required strength of weld in tension

$$f_{wt} := F_{yb} \cdot t_{wb}$$ $f_{wt} = 1.715\ \frac{kN}{mm}$

Nominal strength of weld in tension

$$f_{n.wt} := 2 \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}} \cdot 1.5$$ $f_{n.wt} = 3.681\ \frac{kN}{mm}$

Interaction ratio for weld in tension

$$I_7 := \frac{2.0\ f_{wt}}{f_{n.wt}}$$ $I_7 = 0.932$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws} := d_{xb} - t_{fb} - p_i - 6\ in - k_{des}$$ $l_{ws} = 128.92\ mm$

Nominal strenght of weld in shear

$$V_{n.ws} := 2 \cdot l_{ws} \cdot 0.6 \cdot F_{EXX} \cdot \frac{w_1}{\sqrt{2}}$$ $V_{n.ws} = 316.362\ kN$

Interaction ratio for weld in shear

$$I_8 := \frac{2.0\ V_u}{V_{n.ws}}$$ $I_8 = 0.228$

Column flange flexural yielding

Dimensions

$$s_c := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g}$$ $s_c = 123.085\ mm$

$$c := p_i + p_o + t_{fb}$$ $c = 149.78\ mm$

Yield line parameter for column flange

$$Y_c := \frac{b_{fc}}{2} \cdot \left(\frac{h_1 + h_0}{s}\right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(s + \frac{3\ c}{4}\right) + h_0 \cdot \left(s + \frac{c}{4}\right) + \frac{c^2}{2}\right) + \frac{g}{2}$$

$Y_c = 3351.429\ mm$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 1.67 \cdot M_{np}}{2.0 \cdot F_{yc} \cdot Y_c}}$$ $t_{fc.min} = 27.174\ mm$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}}$$ $I_9 = 0.622$

Column web yielding strength

Web yielding factor

$$C_t := \mathbf{if}\left(a < d_{xc}, 0.5, 1\right)$$ $C_t = 0.5$

Bearing length of beam flange

$$N := t_{fb} + 2 \cdot w_2$$ $N = 17.78\ mm$

Nominal strength in web yielding

$$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t\right) + N\right) \cdot F_{yc} \cdot t_{wc}$$ $R_{n.wy} = \left(1.526 \cdot 10^3\right)\ kN$

Interaction ratio in web yielding

$$I_{10} := \frac{1.5 \cdot P_{uf}}{R_{n.wy}}$$ $I_{10} = 0.939$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + \left(\frac{4 \cdot N}{d_{xc}} - 0.2\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc} := \text{if}\left(\left(a+0.5 \cdot t_{fb}\right) \geq 0.5 \cdot d_{xc}, R_{n.wc1}, \text{if}\left(\left(N \div d_{xc}\right) < 0.2, R_{n.wc2}, R_{n.wc3}\right)\right)$$

$$R_{n.wc} = 2823.519 \ kN$$

Interaction ratio for web crippling

$$I_{11} := \frac{2.0 \ P_{uf}}{R_{n.wc}} \qquad I_{11} = 0.677$$

Column panel shear strength

Axial yield strength of column

$$P_y := F_{yc} \cdot A_c \qquad P_y = 11050 \ kN$$

Panel zone shear strength

$$R_{n.pz1} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right)$$

$$R_{n.pz2} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_{uc}}{P_y}\right)$$

$$R_{n.pz} := \text{if}\left(P_{uc} \leq 0.75 \cdot P_y, R_{n.pz1}, R_{n.pz2}\right) \qquad R_{n.pz} = 2645.601 \ kN$$

Interaction ratio in panel zone shear

$$I_{12} := \frac{1.67 \cdot P_{uf}}{R_{n.pz}} \qquad I_{12} = 0.603$$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min} := \frac{d_{xb} - t_{fb} + d_{xc} - 2 \cdot t_{fc}}{90} \qquad t_{wc.min} = 7.331 \ mm$$

Interaction ratio in web shear buckling

$$I_{13} := \frac{t_{wc.min}}{t_{wc}} \qquad I_{13} = 0.27$$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

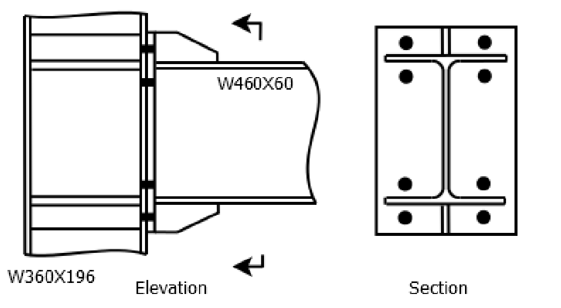
Table 5: Validation problem 4 results

	Interaction Ratio		
Check	Calculated	Osoconn	Result
Bolt tension check	0.988	0.988	OK
Bolt shear check	0.067	0.067	OK
Bolt bearing on end plate check	0.028	0.028	OK
Bolt bearing at column flange	0.044	0.044	OK
Plate thickness check	0.894	0.893	OK
Plate shear yielding check	0.398	0.398	OK
Plate shear rupture check	0.39	0.39	OK
Beam web to end plate weld tension check	0.932	0.932	OK
Beam web to end plate weld shear check	0.228	0.228	OK
Column flange flexural yielding	0.622	0.622	OK
Column web yielding strength	0.939	0.939	OK
Column web crippling check	0.677	0.677	OK
Column panel shear strength	0.603	0.603	OK
Web shear buckling check	0.27	0.27	OK

2.6 Validation Problem 5

Problem Statement

Design a 4-bolt stiffened extended end-plate moment connection for a W460X60 beam framing into the flange of a W360X196 column using the ASD method. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 30kN, and the axial force in the column is 540kN. The beam and column are grade ASTM A992. Plates are of grade ASTM A36. The bolts are ASTM A3125 A490 bearing type (installed with pre-tension).



Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 250\ MPa$
Tensile strength	$F_{up} := 400\ MPa$
Material grade of beam	ASTM A992
Yield strength	$F_{yb} := 345\ MPa$
Tensile strength	$F_{ub} := 450\ MPa$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.1$
Material grade of column	ASTM A992
Yield strength	$F_{yc} := 345\ MPa$
Tensile strength	$F_{uc} := 450\ MPa$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 482\ MPa$
Material specification for bolts	ASTM 3125 A490
Tensile strength	$F_{nt} := 780\ MPa$
Shear strength	$F_{nv} := 469\ MPa$
Young's modulus for steel	$E := 200000\ MPa$

Design Forces

Shear force in beam	$V_u := 30\ kN$
Axial force in column	$P_{uc} := 540\ kN$

Connection Geometry

Beam section	W460X60
Section depth	$d_{xb} := 455\ mm$
Flange width	$b_{fb} := 153\ mm$
Flange thickness	$t_{fb} := 13.3\ mm$
Web thickness	$t_{wb} := 8\ mm$
Design distance from outer face to fillet edge	$k_{des} := 23.5\ mm$
Detailing distance from outer face to fillet edge	$k_{det} := 30.2\ mm$
Plastic section modulus	$Z_x := 1280\ cm^3$
Column section	W360X196
Section depth	$d_{xc} := 373\ mm$
Flange width	$b_{fc} := 373\ mm$
Flange thickness	$t_{fc} := 26.2\ mm$
Web thickness	$t_{wc} := 16.4\ mm$
Cross section area of column	$A_c := 25000\ mm^2$
Design dist form outer face to fillet edge	$k_c := 41.4\ mm$
End-plate width	$B := 400\ mm$
End-plate height	$H := 900\ mm$
End-plate thickness	$t := 35\ mm$
Bolt diameter	$d_b := 30\ mm$
Bolt hole diameter	$d_{bh} := 33\ mm$
Inner bolt pitch	$p_i := 50\ mm$
Outer bolt pitch	$p_o := 160\ mm$
Bolt gage	$g := 150\ mm$
Plate stiffener thickness	$t_{sp} := 16\ mm$
Plate stiffener length	$l_{sp} := 400\ mm$
Web weld thickness	$w_1 := 10\ mm$
Flange weld reinforcement	$w_2 := 8\ mm$
Plate stiffener weld	$w_3 := 12\ mm$
Column stiffener thickness	$t_{st} := 18\ mm$
Width of column stiffener	$B_{st} := 150\ mm$
Column stiffener clip dimension	$k_{st} := 25\ mm$
Thickness of stiffener to flange weld	$t_{w4} := 14\ mm$
Thickness of stiffener to web weld	$t_{w5} := 6\ mm$
Doubler plate thickness	$t_{dp} := 35\ mm$
Doubler plate height	$w_{dp} := 850\ mm$
Doubler plate weld thickness	$t_{w6} := 10\ mm$
Difference between top of beam and top of column	$a := 250\ mm$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 534.336\ kN \cdot m$$

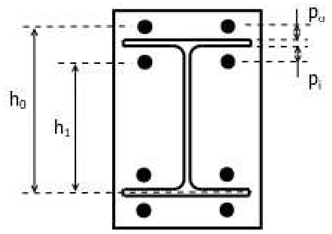
Design moment at face of column

$$M_u := M_p + V_u \cdot (l_{sp} + t) \qquad M_u = 547.386\ kN \cdot m$$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}} \qquad P_{uf} = 1239.271\ kN$$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$$h_0 := d_{xb} - 0.5\ t_{fb} + p_o \qquad h_0 = 608.35\ mm$$

$$h_1 := d_{xb} - t_{fb} - p_i \qquad h_1 = 391.7\ mm$$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2.0 \cdot 2 \cdot M_u}{\pi \cdot F_{nt} \cdot \sum h}} \qquad d_{b.min} = 29.891\ mm$$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b} \qquad I_0 = 0.996$$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot {d_b}^2}{4} \qquad A_b = 706.858\ mm^2$$

Nominal strength of bolt is shear

$$R_{n.bv} := F_{nv} \cdot A_b \qquad R_{n.bv} = 331.517\ kN$$

Interaction ratio in bolt shear

$$I_1 := \frac{2.0 \cdot V_u}{4 \cdot R_{n.bv}} \qquad I_1 = 0.045$$

Bolt bearing on end plate check

Bolt hole edge to edge distance

$$ed_1 := 0.5 \cdot (H - d_{xb}) - p_o - 0.5 \cdot d_{bh} \qquad ed_1 = 46\ mm$$

Bolt hole to hole edge distance

$s_1 := p_i + p_o - d_{bh}$ $\quad s_1 = 177\ mm$

Clear distances for bearing calculation

For outer bolt

$L_{co} := min\left(ed_1, s_1\right)$ $\quad L_{co} = 46\ mm$

For inner bolt

$L_{ci} := s_1$ $\quad L_{ci} = 177\ mm$

Nominal strength in bearing

For outer bolts

$R_{n.bbpo} := 2 \cdot min\left(1.2 \cdot L_{co} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $\quad R_{n.bbpo} = \left(1.546 \cdot 10^3\right)\ kN$

For inner bolts

$R_{n.bbpi} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $\quad R_{n.bbpi} = 2016\ kN$

Net bearing strength

$R_{n.bbp} := R_{n.bbpi} + R_{n.bbpo}$ $\quad R_{n.bbp} = 3561.6\ kN$

Interaction ratio in bearing at end plate

$I_2 := \dfrac{2.0 \cdot V_u}{R_{n.bbp}}$ $\quad I_2 = 0.017$

Bolt bearing at column flange

Bolt hole edge to edge of column

$ed_2 := a - p_o - 0.5\ d_{bh}$ $\quad ed_2 = 73.5\ mm$

Clear distances for bearing calculation

For outer bolt

$L_{co2} := min\left(ed_2, s_1\right)$ $\quad L_{co} = 46\ mm$

Nominal strength in bearing

For outer bolts

$R_{n.bbco} := 2 \cdot min\left(1.2 \cdot L_{co2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $\quad R_{n.bbco} = 1697.76\ kN$

For inner bolts

$R_{n.bbci} := 2 \cdot min\left(1.2 \cdot L_{ci} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $\quad R_{n.bbci} = 1697.76\ kN$

Net bearing strength

$R_{n.bbc} := R_{n.bbci} + R_{n.bbco}$ $\quad R_{n.bbc} = 3395.52\ kN$

Interaction ratio in bearing at end plate

$I_3 := \dfrac{2.0 \cdot V_u}{R_{n.bbc}}$ $\quad I_3 = 0.018$

Plate thickness check

Bolt tension strength

$P_t := F_{nt} \cdot A_b$ $\quad P_t = 551.35\ kN$

No-prying moment strength of plate

$$M_{np}:=2\cdot P_t\cdot \sum h \qquad M_{np}=\left(1.103\cdot 10^{3}\right)\ kN\cdot m$$

Dimension

$$s:=\frac{1}{2}\cdot\sqrt{B\cdot g} \qquad s=122.474\ mm$$

$$p'_i:=min\left(p_i,s\right) \qquad p'_i=50\ mm$$

Vertical edge distance of outer bolt

$$d_e:=\frac{H-d_{xb}-2\ p_o}{2} \qquad d_e=62.5\ mm$$

End plate yield line parameter

$$Y_{p1}:=\frac{B}{2}\cdot\left(h_1\cdot\left(\frac{1}{p_i}+\frac{1}{s}\right)+h_0\cdot\left(\frac{1}{p_o}+\frac{1}{2\ s}\right)\right)+\frac{2}{g}\left(h_1\cdot\left(p_i+s\right)+h_0\cdot\left(d_e+p_o\right)\right)$$

$$Y_{p2}:=\frac{B}{2}\cdot\left(h_1\cdot\left(\frac{1}{p_i}+\frac{1}{s}\right)+h_0\cdot\left(\frac{1}{p_o}+\frac{1}{s}\right)\right)+\frac{2}{g}\left(h_1\cdot\left(p_i+s\right)+h_0\cdot\left(s+p_o\right)\right)$$

$$Y_p:=\mathbf{if}\left(d_e<s,Y_{p1},Y_{p2}\right) \qquad Y_p=6169.145\ mm$$

Required end plate thickness

$$t_{p.min}:=\sqrt{\frac{1.11\cdot 1.67\cdot M_{np}}{2.0\cdot F_{yp}\cdot Y_p}} \qquad t_{p.min}=25.743\ mm$$

Interaction ratio for plate thickness

$$I_4:=\frac{t_{p.min}}{t} \qquad I_4=0.736$$

Beam web to plate weld tension check

Required strength of weld in tension

$$f_{wt}:=F_{yb}\cdot t_{wb} \qquad f_{wt}=2.76\ \frac{kN}{mm}$$

Nominal strength of weld in tension

$$f_{n.wt}:=2\cdot 0.6\cdot F_{EXX}\cdot\frac{w_1}{\sqrt{2}}\cdot 1.5 \qquad f_{n.wt}=6.135\ \frac{kN}{mm}$$

Interaction ratio for weld in tension

$$I_5:=\frac{2.0\ f_{wt}}{f_{n.wt}} \qquad I_5=0.9$$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws}:=d_{xb}-t_{fb}-p_i-6\ in-k_{des} \qquad l_{ws}=215.8\ mm$$

Nominal strenght of weld in shear

$$V_{n.ws}:=2\cdot l_{ws}\cdot 0.6\cdot F_{EXX}\cdot\frac{w_1}{\sqrt{2}} \qquad V_{n.ws}=882.602\ kN$$

Interaction ratio for weld in shear

$$I_6 := \frac{2.0\ V_u}{V_{n.ws}} \qquad I_6 = 0.068$$

End plate stiffener thickness check

Height of stiffener

$$h_{sp} := \frac{H - d_{xb}}{2} \qquad h_{sp} = 222.5\ mm$$

Minimum thickness for stiffener buckling

$$t_{sp.buc} := 1.79 \cdot h_{sp} \cdot \sqrt{\frac{F_{yp}}{E}} \qquad t_{sp.buc} = 14.081\ mm$$

Minimum required thickness for stiffener

$$t_{sp.min} := \max\left(t_{sp.buc}, t_{wb} \cdot \left(\frac{F_{yb}}{F_{yp}}\right)\right) \qquad t_{sp.min} = 14.081\ mm$$

Interaction ratio for stiffener thickness

$$I_7 := \frac{t_{sp.min}}{t_{sp}} \qquad I_7 = 0.88$$

Plate stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w3} := 0.6\ F_{EXX} \cdot \sqrt{2} \cdot w_3 \qquad R_{n.w3} = 4.908\ \frac{kN}{mm}$$

Interaction ratio in stiffener buckling

$$I_8 := \frac{2.0 \cdot 0.6 \cdot F_{yp} \cdot t_{sp}}{R_{n.w3}} \qquad I_8 = 0.978$$

Column flange flexural yielding

Dimensions

$$s_c := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g} \qquad s_c = 118.269\ mm$$

$$p_{si} := p_i + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{si} = 47.65\ mm$$

$$p_{so} := p_o + 0.5 \cdot t_{fb} - 0.5 \cdot t_{st} \qquad p_{so} = 157.65\ mm$$

Yield line parameter for column flange

$$Y_c := \frac{b_{fc}}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s} + \frac{1}{p_{si}}\right) + h_0 \cdot \left(\frac{1}{s} + \frac{1}{p_{so}}\right)\right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(s + p_{si}\right) + h_0 \cdot \left(s + p_{so}\right)\right)$$

$$Y_c = 6936.304\ mm$$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 1.67 \cdot M_{np}}{2.0 \cdot F_{yc} \cdot Y_c}} \qquad t_{fc.min} = 20.667\ mm$$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}} \qquad I_9 = 0.789$$

Column web yielding strength

Web yielding factor

$$C_t := \text{if}\left(a < d_{xc}, 0.5, 1\right) \qquad C_t = 0.5$$

Bearing length of beam flange

$$N := t_{fb} + 2 \cdot w_2 \qquad N = 29.3\ mm$$

Nominal strength in web yielding

$$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t\right) + N\right) \cdot F_{yc} \cdot t_{wc} \qquad R_{n.wy} = 1066.533\ kN$$

Interaction ratio in web yielding

$$I_{10} := \frac{1.5 \cdot P_{uf}}{R_{n.wy}} \qquad I_{10} := 0$$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + \left(\frac{4 \cdot N}{d_{xc}} - 0.2\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc} := \text{if}\left(\left(a + 0.5 \cdot t_{fb}\right) \ge 0.5 \cdot d_{xc}, R_{n.wc1}, \text{if}\left(\left(N \div d_{xc}\right) < 0.2, R_{n.wc2}, R_{n.wc3}\right)\right)$$

$$R_{n.wc} = 2522.725\ kN$$

Interaction ratio for web crippling

$$I_{11} := \frac{2.0 \cdot P_{uf}}{R_{n.wc}} \qquad I_{11} := 0$$

Column panel shear strength

Axial yield strength of column

$$P_y := F_{yc} \cdot A_c \qquad P_y = 8625\ kN$$

Panel zone shear strength

$$R_{n.pz1} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right)$$

$$R_{n.pz2} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_{uc}}{P_y}\right)$$

$$R_{n.pz} := \text{if}\left(P_{uc} \le 0.75 \cdot P_y, R_{n.pz1}, R_{n.pz2}\right) \qquad R_{n.pz} = 1615.716\ kN$$

Interaction ratio in panel zone shear

$$I_{12} := \frac{1.67 \cdot P_{uf}}{R_{n.pz}} \qquad I_{12} := 0$$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min} := \frac{d_{xb} - t_{fb} + d_{xc} - 2 \cdot t_{fc}}{90} \qquad t_{wc.min} = 8.47\ mm$$

Interaction ratio in web shear buckling

$$I_{13} := \frac{t_{wc.min}}{t_{wc}} \qquad I_{13} = 0.516$$

Transverse stiffener checks

Maximum flange force that can be delivered to unstiffened column flange

$$R_{n.fy} := \frac{F_{yc} \cdot Y_c \cdot t_{fc}^{\ 2}}{d_{xb} - t_{fb}} \qquad R_{n.fy} = 3718.967\ kN$$

Required strength of transverse stiffeners

$$R_{u.st} := P_{uf} - min\left(\frac{R_{n.fy}}{1.67}, \frac{R_{n.wy}}{1.5}, \frac{R_{n.wc}}{2.0}\right) \qquad R_{u.st} = 528.249\ kN$$

Transverse stiffener yielding check

Nominal strength of striffener in yielding

$$R_{n.sty} := 2 \cdot F_{yp} \cdot t_{st} \cdot \left(B_{st} - k_{st}\right) \qquad R_{n.sty} = \left(1.125 \cdot 10^3\right)\ kN$$

Interaction ratio for stiffener yielding

$$I_{14} := \frac{1.67 \cdot R_{u.st}}{R_{n.sty}} \qquad I_{14} = 0.784$$

Transverse stiffener shear check

Nominal strength of stiffener in shear

$$R_{n.stv} := 0.6 \cdot F_{yp} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot t_{st} \qquad R_{n.stv} = 730.62\ kN$$

Interaction ratio for stiffener in shear

$$I_{15} := \frac{1.67 \cdot R_{u.st}}{2 \cdot R_{n.stv}} \qquad I_{15} = 0.604$$

Transverse stiffener thickness check

Minimum thickness of stiffener

$$t_{st.min} := \max\left(\frac{\frac{B_{st}}{in} \cdot \sqrt{\frac{F_{yp}}{ksi}}}{95}\ in, \frac{t_{fb}}{2}\right) \qquad t_{st.min} = 9.508\ mm$$

Interaction ratio for stiffener thickness

$$I_{16} := \frac{t_{st.min}}{t_{st}} \qquad I_{16} = 0.528$$

Transverse stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w4} := 1.5 \cdot 0.6\ F_{EXX} \cdot \sqrt{2} \cdot t_{w4} \qquad R_{n.w4} = 8.589\ \frac{kN}{mm}$$

Interaction ratio for weld

$$I_{17} := \frac{2.0 \cdot 0.9\ F_{yp} \cdot t_{st}}{R_{n.w4}} \qquad I_{17} = 0.943$$

Transverse stiffener to web weld check

Nominal strength of stiffener to web weld

$$R_{n.w5} := 2 \cdot 0.6\ F_{EXX} \cdot \left(d_{xc} - 2 \cdot t_{fc} - 2 \cdot k_{st}\right) \cdot \sqrt{2} \cdot t_{w5} \qquad R_{n.w5} = 1328.074\ kN$$

Interaction ratio for stiffener to web weld

$$I_{18} := \frac{2.0\ R_{u.st}}{R_{n.w5}} \qquad I_{18} = 0.796$$

Doubler plate shear check

Required strength for doubler plate

$$R_{u.dp} := P_{uf} - \frac{R_{n.pz}}{1.67} \qquad R_{u.dp} = 271.777\ kN$$

Nominal strength of doubler plate in shear

$$R_{n.dp} := 0.6 \cdot F_{yp} \cdot d_{xc} \cdot t_{dp} \qquad R_{n.dp} = 1958.25\ kN$$

Interaction ratio for double plate shear

$$I_{19} := \frac{1.5 \cdot R_{u.dp}}{R_{n.dp}} \qquad I_{19} = 0.208$$

Doubler plate width check

Minimum width of the doubler plate

$$w_{dp.min} := d_{xb} + 2 \cdot \left(3\ k_c + t\right) \qquad w_{dp.min} = 773.4\ mm$$

Interaction for doubler plate width

$$I_{20} := \frac{w_{dp.min}}{w_{dp}} \qquad I_{20} = 0.91$$

Doubler plate thickness check

Minimum thickness of doubler plate

$$t_{dp.min1} := \frac{\left(d_{xb} - t_{fb}\right) - t_{st} + d_{xc} - 2\ t_{fc}}{90}$$

$$t_{dp.min2} := \frac{\left(d_{xc} - 2\ k_c\right) \cdot \sqrt{\frac{F_{yp}}{ksi}}}{418}$$

$$t_{dp.min} := \max\left(t_{dp.min1}, t_{dp.min2}\right) \qquad t_{dp.min} = 8.27\ mm$$

Interaction ratio for doubler plate thickness

$$I_{21} := \frac{t_{dp.min}}{t_{dp}}$$ $I_{21} = 0.236$

Doubler plate to flange weld check

Nominal strength of fillet weld

$$R_{n.w6} := 0.6 \cdot F_{EXX} \cdot \frac{1}{\sqrt{2}} \cdot t_{w6} \cdot w_{dp}$$ $R_{n.w6} = 1738.21\ kN$

Interaction ratio for flange weld

$$I_{22} := \frac{2.0\ R_{u.dp}}{R_{n.w6}}$$ $I_{22} = 0.313$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

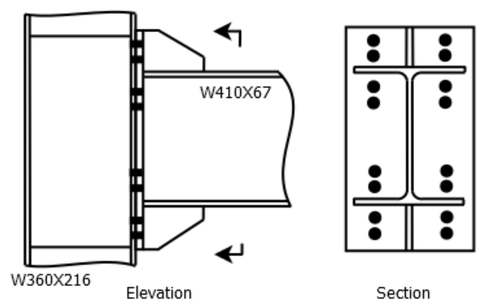
Table 6: Validation problem 5 results

Check	Interaction Ratio		Result
	Calculated	Osoconn	
Bolt tension check	0.996	0.996	OK
Bolt shear check	0.045	0.045	OK
Bolt bearing on end plate check	0.017	0.017	OK
Bolt bearing at column flange	0.018	0.018	OK
Plate thickness check	0.736	0.735	OK
Beam web to plate weld tension check	0.9	0.9	OK
Beam web to plate weld shear check	0.068	0.068	OK
End plate stiffener thickness check	0.88	0.88	OK
End plate stiffener to flange weld check	0.978	0.978	OK
Column flange flexural yielding	0.789	0.789	OK
Column web yielding strength	0.0	0.0	OK
Column web crippling check	0.0	0.0	OK
Column panel shear strength	0.0	0.0	OK
Web shear buckling check	0.516	0.516	OK
Transverse stiffener yielding check	0.784	0.784	OK
Transverse stiffener shear check	0.604	0.604	OK
Transverse stiffener thickness check	0.528	0.528	OK
Transverse stiffener to flange weld check	0.943	0.943	OK
Transverse stiffener to web weld check	0.796	0.796	OK
Doubler plate shear check	0.208	0.208	OK
Doubler plate width check	0.91	0.91	OK
Doubler plate thickness check	0.236	0.236	OK
Doubler plate to flange weld check	0.313	0.313	OK

2.7 Validation Problem 6

Problem Statement

Design a 8-bolt stiffened extended end-plate moment connection for a W410X67 beam framing into the flange of a W360X216 column using the LRFD method. The connection shall designed and detailed to resist seismic forces. The connection is subjected to a shear forces of 510kN, and the axial force in the column is 8050kN. The beam and column are grade ASTM A992. Plates are of grade ASTM A36. The bolts are ASTM A3125 A490 bearing type (installed with pre-tension.)



Design Inputs

Material Properties

Material grade for plate	ASTM A36
Yield strength	$F_{yp} := 250\ MPa$
Tensile strength	$F_{up} := 400\ MPa$
Material grade of beam	ASTM A992
Yield strength	$F_{yb} := 345\ MPa$
Tensile strength	$F_{ub} := 450\ MPa$
Ratio of the expected yield strength to specified yield strength	$R_y := 1.1$
Material grade of column	ASTM A992
Yield strength	$F_{yc} := 345\ MPa$
Tensile strength	$F_{uc} := 450\ MPa$
Material grade for weld electrode	E70XX
Tensile strength	$F_{EXX} := 482\ MPa$
Material specification for bolts	ASTM 3125 A490
Tensile strength	$F_{nt} := 780\ MPa$
Shear strength	$F_{nv} := 469\ MPa$
Young's modulus for steel	$E := 200000\ MPa$

Design Forces

Shear force in beam	$V_u := 510\ kN$
Axial force in column	$P_{uc} := 8050\ kN$

Connection Geometry

Beam section W410X67

Section depth $d_{xb} := 409\ mm$

Flange width $b_{fb} := 179\ mm$

Flange thickness $t_{fb} := 14.4\ mm$

Web thickness $t_{wb} := 8.76\ mm$

Design distance from outer face to fillet edge $k_{des} := 24.6\ mm$

Detailing distance from outer face to fillet edge $k_{det} := 31.8\ mm$

Plastic section modulus $Z_x := 1350\ cm^3$

Column section W360X216

Section depth $d_{xc} := 376\ mm$

Flange width $b_{fc} := 394\ mm$

Flange thickness $t_{fc} := 27.7\ mm$

Web thickness $t_{wc} := 17.3\ mm$

Cross section area of column $A_c := 27500\ mm^2$

Design dist form outer face to fillet edge $k_c := 42.9\ mm$

End-plate width $B := 400\ mm$

End-plate height $H := 700\ mm$

End-plate thickness $t := 35\ mm$

Bolt diameter $d_b := 24\ mm$

Bolt hole diameter $d_{bh} := 27\ mm$

Inner bolt pitch $p_i := 40\ mm$

Outer bolt pitch $p_o := 40\ mm$

Bolt to bolt pitch $p_b := 65\ mm$

Bolt gage $g := 140\ mm$

Plate stiffener thickness $t_{sp} := 16\ mm$

Plate stiffener length $l_{sp} := 300\ mm$

Web weld thickness $w_1 := 12\ mm$

Flange weld reinforcement $w_2 := 8\ mm$

Plate stiffener weld $w_3 := 8\ mm$

Doubler plate thickness $t_{dp} := 35\ mm$

Doubler plate width $w_{dp} := 850\ mm$

Doubler plate weld thickness $t_{w6} := 10\ mm$

Difference between top of beam and top of column $a := 900\ mm$

Design Calculations

Expected plastic moment

$$M_p := 1.1 \cdot R_y \cdot F_{yb} \cdot Z_x \qquad M_p = 563.558\ kN \cdot m$$

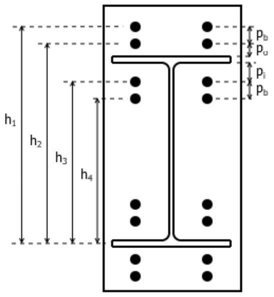
Design moment at face of column

$$M_u := M_p + V_u \cdot (l_{sp} + t) \qquad M_u = 734.408\ kN \cdot m$$

Flange force

$$P_{uf} := \frac{M_u}{d_{xb} - t_{fb}}$$ $$P_{uf} = 1861.144\ kN$$

Bolt tension check



Distance of centreline of compression flange to tension bolt

$$h_1 := d_{xb} - 0.5\ t_{fb} + p_o + p_b$$ $$h_1 = 506.8\ mm$$

$$h_2 := h_1 - p_b$$ $$h_2 = 441.8\ mm$$

$$h_3 := d_{xb} - t_{fb} - p_i$$ $$h_3 = 354.6\ mm$$

$$h_4 := h_3 - p_b$$ $$h_4 = 289.6\ mm$$

Minimum required bolt diameter

$$d_{b.min} := \sqrt{\frac{2 \cdot M_u}{\pi \cdot 0.75 \cdot F_{nt} \cdot \sum h}}$$ $$d_{b.min} = 22.4\ mm$$

Interaction ratio for bolts in tension

$$I_0 := \frac{d_{b.min}}{d_b}$$ $$I_0 = 0.933$$

Bolt shear check

Area of bolt

$$A_b := \frac{\pi \cdot d_b^2}{4}$$ $$A_b = 452.389\ mm^2$$

Nominal strength of bolt is shear

$$R_{n.bv} := F_{nv} \cdot A_b$$ $$R_{n.bv} = 212.171\ kN$$

Interaction ratio in bolt shear

$$I_1 := \frac{V_u}{8 \cdot 0.75 \cdot R_{n.bv}}$$ $$I_1 = 0.401$$

Bolt bearing on end plate check

Vertical edge distance of outer bolt

$$d_e := \frac{H - d_{xb} - 2\ (p_o + p_b)}{2}$$ $$d_e = 40.5\ mm$$

Clear distances for bearing calculation for each bolt row

$L_{c1} := min\left(d_e - 0.5\ d_{bh}, p_b - d_{bh}\right)$ $L_{c1} = 27\ mm$

$L_{c2} := min\left(p_b - d_{bh}, p_o + p_i + t_{fb} - d_{bh}\right)$ $L_{c2} = 38\ mm$

$L_{c3} := L_{c2}$ $L_{c3} = 38\ mm$

$L_{c4} := p_b - d_{bh}$ $L_{c4} = 38\ mm$

Nominal strength in bearing for each bolt row

$R_{n.bbp1} := 2 \cdot min\left(1.2 \cdot L_{c1} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbp1} = 907.2\ kN$

$R_{n.bbp2} := 2 \cdot min\left(1.2 \cdot L_{c2} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbp2} = 1276.8\ kN$

$R_{n.bbp3} := 2 \cdot min\left(1.2 \cdot L_{c3} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbp3} = 1276.8\ kN$

$R_{n.bbp4} := 2 \cdot min\left(1.2 \cdot L_{c4} \cdot t \cdot F_{up}, 2.4 \cdot d_b \cdot t \cdot F_{up}\right)$ $R_{n.bbp4} = 1276.8\ kN$

Net bearing strength

$R_{n.bbp} := R_{n.bbp1} + R_{n.bbp2} + R_{n.bbp3} + R_{n.bbp4}$ $R_{n.bbp} = 4737.6\ kN$

Interaction ratio in bearing at end plate

$I_2 := \frac{V_u}{0.75 \cdot R_{n.bbp}}$ $I_2 = 0.144$

Bolt bearing at column flange

Clear distances for bearing calculation of outer bolt

$L_{c1} := min\left(a - p_o - p_b - 0.5\ d_{bh}, p_b - d_{bh}\right)$ $L_{c1} = 38\ mm$

Nominal strength in bearing for each bolt row

$R_{n.bbc1} := 2 \cdot min\left(1.2 \cdot L_{c1} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbc1} = 1136.808\ kN$

$R_{n.bbc2} := 2 \cdot min\left(1.2 \cdot L_{c2} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbc1} = 1136.808\ kN$

$R_{n.bbc3} := 2 \cdot min\left(1.2 \cdot L_{c3} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbc1} = 1136.808\ kN$

$R_{n.bbc4} := 2 \cdot min\left(1.2 \cdot L_{c4} \cdot t_{fc} \cdot F_{uc}, 2.4 \cdot d_b \cdot t_{fc} \cdot F_{uc}\right)$ $R_{n.bbc1} = 1136.808\ kN$

Net bearing strength

$R_{n.bbc} := R_{n.bbc1} + R_{n.bbc2} + R_{n.bbc3} + R_{n.bbc4}$ $R_{n.bbc} = 4547.232\ kN$

Interaction ratio in bearing at end plate

$I_3 := \frac{V_u}{0.75 \cdot R_{n.bbc}}$ $I_3 = 0.15$

Plate thickness check

Bolt tension strength

$P_t := F_{nt} \cdot A_b$ $P_t = 352.864\ kN$

No-prying moment strength of plate

$M_{np} := 2 \cdot P_t \cdot \sum h$ $M_{np} = 1124.083\ kN \cdot m$

Dimension

$$s:=\frac{1}{2}\cdot\sqrt{B\cdot g}$$ $s=118.322\ mm$

$$p'_i:=min\left(p_i,s\right)$$ $p'_i=40\ mm$

End plate yield line parameter

$$Y_{p1}:=\frac{B}{2}\cdot\left(h_1\cdot\left(\frac{1}{2\ d_e}\right)+h_2\cdot\left(\frac{1}{p_o}\right)+h_3\cdot\left(\frac{1}{p_i}\right)+h_4\cdot\left(\frac{1}{s}\right)\right)$$

$$Y_{p1}:=Y_{p1}+\frac{2}{g}\left(h_1\cdot\left(d_e+\frac{p_b}{4}\right)+h_2\cdot\left(p_o+\frac{3\ p_b}{4}\right)+h_3\cdot\left(p_i+\frac{p_b}{4}\right)+h_4\cdot\left(s+\frac{3\ p_b}{4}\right)+p_b^{\ 2}\right)+g$$

$$Y_{p2}:=\frac{B}{2}\cdot\left(h_1\cdot\left(\frac{1}{s}\right)+h_2\cdot\left(\frac{1}{p_o}\right)+h_3\cdot\left(\frac{1}{p_i}\right)+h_4\cdot\left(\frac{1}{s}\right)\right)$$

$$Y_{p2}:=Y_{p2}+\frac{2}{g}\left(h_1\cdot\left(s+\frac{p_b}{4}\right)+h_2\cdot\left(p_o+\frac{3\ p_b}{4}\right)+h_3\cdot\left(p_i+\frac{p_b}{4}\right)+h_4\cdot\left(s+\frac{3\ p_b}{4}\right)+p_b^{\ 2}\right)+g$$

$$Y_p:=\text{if}\left(d_e<s,Y_{p1},Y_{p2}\right)$$ $Y_p=7870.383\ mm$

Required end plate thickness

$$t_{p.min}:=\sqrt{\frac{1.11\cdot0.75\cdot M_{np}}{0.9\cdot F_{yp}\cdot Y_p}}$$ $t_{p.min}=22.988\ mm$

Interaction ratio for plate thickness

$$I_4:=\frac{t_{p.min}}{t}$$ $I_4=0.657$

Beam web to plate weld tension check

Required strength of weld in tension

$$f_{wt}:=F_{yb}\cdot t_{wb}$$ $f_{wt}=17.257\ \frac{kip}{in}$

Nominal strength of weld in tension

$$f_{n.wt}:=2\cdot0.6\cdot F_{EXX}\cdot\frac{w_1}{\sqrt{2}}\cdot1.5$$ $f_{n.wt}=42.037\ \frac{kip}{in}$

Interaction ratio for weld in tension

$$I_5:=\frac{f_{wt}}{0.75\cdot f_{n.wt}}$$ $I_5=0.547$

Beam web to plate weld shear check

Length of web weld assumed to resist shear

$$l_{ws}:=d_{xb}-t_{fb}-p_i-6\ in-k_{des}$$ $l_{ws}=6.992\ in$

Nominal strenght of weld in shear

$$V_{n.ws}:=2\cdot l_{ws}\cdot0.6\cdot F_{EXX}\cdot\frac{w_1}{\sqrt{2}}$$ $V_{n.ws}=195.953\ kip$

Interaction ratio for weld in shear

$$I_6 := \frac{V_u}{0.75 \cdot V_{n.ws}}$$ $$I_6 = 0.78$$

End plate stiffener thickness check

Height of stiffener

$$h_{sp} := \frac{H - d_{xb}}{2}$$ $$h_{sp} = 145.5\ mm$$

Minimum thickness for stiffener buckling

$$t_{sp.buc} := 1.79 \cdot h_{sp} \cdot \sqrt{\frac{F_{yp}}{E}}$$ $$t_{sp.buc} = 9.208\ mm$$

Minimum required thickness for stiffener

$$t_{sp.min} := \max\left(t_{sp.buc}, t_{wb} \cdot \left(\frac{F_{yb}}{F_{yp}}\right)\right)$$ $$t_{sp.min} = 12.089\ mm$$

Interaction ratio for stiffener thickness

$$I_7 := \frac{t_{sp.min}}{t_{sp}}$$ $$I_7 = 0.756$$

End plate stiffener to flange weld check

Nominal strength of stiffener to flange weld

$$R_{n.w3} := 0.6\ F_{EXX} \cdot \sqrt{2} \cdot w_3$$ $$R_{n.w3} = 3.272\ \frac{kN}{mm}$$

Interaction ratio in stiffener buckling

$$I_8 := \frac{0.6 \cdot F_{yp} \cdot t_{sp}}{0.75 \cdot R_{n.w3}}$$ $$I_8 = 0.978$$

Column flange flexural yielding

Dimensions

$$s_c := \frac{1}{2} \cdot \sqrt{b_{fc} \cdot g}$$ $$s_c = 117.431\ mm$$

$$c := p_i + p_o + t_{fb}$$ $$c = 94.4\ mm$$

Yield line parameter for column flange

$$Y_c := \frac{b_{fc}}{2} \cdot \left(h_1 \cdot \left(\frac{1}{s}\right) + h_4 \cdot \left(\frac{1}{s}\right)\right) + \frac{2}{g} \cdot \left(h_1 \cdot \left(p_b + \frac{c}{2} + s\right) + h_2 \cdot \left(\frac{p_b}{2} + \frac{c}{4}\right) + h_3 \cdot \left(\frac{p_b}{2} + \frac{c}{2}\right) + h_4 \cdot s\right) + \frac{g}{2}$$

$$Y_c = 431.227\ cm$$

Required column flange thickness

$$t_{fc.min} := \sqrt{\frac{1.11 \cdot 0.75 \cdot M_{np}}{0.9 \cdot F_{yc} \cdot Y_c}}$$ $$t_{fc.min} = 26.437\ mm$$

Interaction ratio for flange yielding

$$I_9 := \frac{t_{fc.min}}{t_{fc}}$$ $$I_9 = 0.954$$

Column web yielding strength

Web yielding factor

$C_t := \text{if}\left(a < d_{xc}, 0.5, 1\right)$ $\qquad C_t = 1$

Bearing length of beam flange

$N := t_{fb} + 2 \cdot w_2$ $\qquad N = 30.4\ mm$

Nominal strength in web yielding

$R_{n.wy} := \left(C_t \cdot \left(6 \cdot k_c + 2 \cdot t\right) + N\right) \cdot F_{yc} \cdot t_{wc}$ $\qquad R_{n.wy} = 2135.529\ kN$

Interaction ratio in web yielding

$$I_{10} := \frac{P_{uf}}{1.0 \cdot R_{n.wy}} \qquad I_{10} = 0.872$$

Column web crippling check

Nominal strength in web crippling

$$R_{n.wc1} := 0.8 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc2} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + 3 \cdot \left(\frac{N}{d_{xc}}\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc3} := 0.4 \cdot t_{wc}^{\ 2} \cdot \left(1 + \left(\frac{4 \cdot N}{d_{xc}} - 0.2\right) \cdot \left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yc} \cdot t_{fc}}{t_{wc}}}$$

$$R_{n.wc} := \text{if}\left(\left(a + 0.5 \cdot t_{fb}\right) \ge 0.5 \cdot d_{xc}, R_{n.wc1}, \text{if}\left(\left(N \div d_{xc}\right) < 0.2, R_{n.wc2}, R_{n.wc3}\right)\right)$$

$R_{n.wc} = 2817.941\ kN$

Interaction ratio for web crippling

$$I_{11} := \frac{P_{uf}}{0.75 \cdot R_{n.wc}} \qquad I_{11} = 0.881$$

Column panel shear strength

Axial yield strength of column

$P_y := F_{yc} \cdot A_c$ $\qquad P_y = 9487.5\ kN$

Panel zone shear strength

$$R_{n.pz1} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right)$$

$$R_{n.pz2} := 0.6 \cdot F_{yc} \cdot d_{xc} \cdot t_{wc} \cdot \left(1 + \frac{3 \cdot b_{fc} \cdot t_{fc}^{\ 2}}{d_{xb} \cdot d_{xc} \cdot t_{wc}}\right) \cdot \left(1.9 - \frac{1.2 \cdot P_{uc}}{P_y}\right)$$

$R_{n.pz} := \text{if}\left(P_{uc} \le 0.75 \cdot P_y, R_{n.pz1}, R_{n.pz2}\right)$ $\qquad R_{n.pz} = 1592.128\ kN$

Interaction ratio in panel zone shear

$$I_{12} := \frac{P_{uf}}{0.9 \cdot R_{n.pz}} \qquad I_{12} := 0$$

Web shear buckling check

Minimum web thickness to prevent shear buckling

$$t_{wc.min} := \frac{d_{xb} - t_{fb} + d_{xc} - 2 \cdot t_{fc}}{90}$$ $t_{wc.min} = 7.947\ mm$

Interaction ratio in web shear buckling

$$I_{13} := \frac{t_{wc.min}}{t_{wc}}$$ $I_{13} = 0.459$

Doubler plate shear check

Required strength for doubler plate

$$R_{u.dp} := P_{uf} - 0.9\ R_{n.pz}$$ $R_{u.dp} = 428.229\ kN$

Nominal strength of doubler plate in shear

$$R_{n.dp} := 0.6 \cdot F_{yp} \cdot d_{xc} \cdot t_{dp}$$ $R_{n.dp} = 1974\ kN$

Interaction ratio for double plate shear

$$I_{14} := \frac{R_{u.dp}}{1.0\ R_{n.dp}}$$ $I_{14} = 0.217$

Doubler plate width check

Minimum width of the doubler plate

$$w_{dp.min} := d_{xb} + 2 \cdot (3\ k_c + t)$$ $w_{dp.min} = 736.4\ mm$

Interaction for doubler plate width

$$I_{15} := \frac{w_{dp.min}}{w_{dp}}$$ $I_{15} = 0.866$

Doubler plate thickness check

Minimum thickness of doubler plate

$$t_{dp.min1} := \frac{(d_{xb} - t_{fb}) + d_{xc} - 2\ t_{fc}}{90}$$

$$t_{dp.min2} := \frac{(d_{xc} - 2\ k_c) \cdot \sqrt{\frac{F_{yp}}{ksi}}}{418}$$

$$t_{dp.min} := \max(t_{dp.min1}, t_{dp.min2})$$ $t_{dp.min} = 7.947\ mm$

Interaction ratio for doubler plate thickness

$$I_{16} := \frac{t_{dp.min}}{t_{dp}}$$ $I_{16} = 0.227$

Doubler plate to flange weld check

Nominal strength of fillet weld

$$R_{n.w6} := 0.6 \cdot F_{EXX} \cdot \frac{1}{\sqrt{2}} \cdot t_{w6} \cdot w_{dp}$$ $R_{n.w6} = 1738.21\ kN$

Interaction ratio for flange weld

$$I_{17} := \frac{R_{u.dp}}{0.75\ R_{n.w6}}$$

$$I_{17} = 0.328$$

Validation Results

The calculated ratios are compared with the output of Osoconn and if it is within a tolerance of 1% the result is deemed to be OK.

Table 7: Validation problem 6 results

	Interaction Ratio		
Check	Calculated	Osoconn	Result
Bolt tension check	0.933	0.933	OK
Bolt shear check	0.401	0.401	OK
Bolt bearing on end plate check	0.144	0.144	OK
Bolt bearing at column flange	0.15	0.15	OK
Plate thickness check	0.657	0.657	OK
Beam web to plate weld tension check	0.547	0.547	OK
Beam web to plate weld shear check	0.78	0.78	OK
End plate stiffener thickness check	0.756	0.756	OK
End plate stiffener to flange weld check	0.978	0.978	OK
Column flange flexural yielding	0.954	0.954	OK
Column web yielding strength	0.872	0.872	OK
Column web crippling check	0.881	0.881	OK
Column panel shear strength	0.0	0.0	OK
Web shear buckling check	0.459	0.459	OK
Doubler plate shear check	0.217	0.217	OK
Doubler plate height check	0.866	0.866	OK
Doubler plate thickness check	0.227	0.227	OK
Doubler plate to flange weld check	0.328	0.329	OK

3 Osoconn Output

3.1 Validation problem 1

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_1
-----------------------------------------------------------------------+---------------------
Design Summary                                                         |
-----------------------------------------------------------------------+---------------------
   Connection is OK                                                    |
   Maximum interaction ratio                                           | 0.972
-----------------------------------------------------------------------+---------------------
Design Inputs                                                          |
-----------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                      | 4 BOLT - UNSTIFFENED
   Design method                                                       | LRFD
   Shear force in connection (V)                                       | 35.000 kip
   Column axial force                                                  | 150.000 kip
   Modulus of elasticity of steel                                      | 29000.000 ksi
   Yield strength of beam (Fyb)                                        | 36.000 ksi
   Tensile strength of beam                                            | 58.000 ksi
   Yield strength of column                                            | 36.000 ksi
   Tensile strength of column                                          | 58.000 ksi
   Yield strength of plate                                             | 36.000 ksi
   Tensile strength of plate                                           | 58.000 ksi
   Tensile strength of weld                                            | 70.000 ksi
   Material overstrength factor for beam (Ry)                          | 1.500
   End plate dimensions (b x d x tp)                                   | 12 x 22 x 1 in
   Bolt diamter                                                        | 1.125 in
   Bolt grade                                                          | ASTM A325
   Thickness of beam web to plate fillet weld                          | 0.250 in
   Thickness of flange weld reinforing fillet weld                     | 0.160 in
   Beam section property                                               | W14X30
      Depth (d)                                                        | 13.800 in
      Flange width                                                     | 6.730 in
      Web thickness                                                    | 0.270 in
      Flange thickness (tf)                                            | 0.385 in
      Fillet dimension                                                 | 0.785 in
      Plastic section modulus (Zx)                                     | 47.300 in^3
   Column section property                                             | W16X100
      Depth                                                            | 17.000 in
      Flange width                                                     | 10.400 in
      Web thickness                                                    | 0.585 in
      Flange thickness                                                 | 0.985 in
      Fillet dimension                                                 | 1.390 in
   Thickness of column stiffener (tsc)                                 | 1.000 in
   Width of column stiffener                                           | 4.000 in
   Column stiffener clip dimension                                     | 1.000 in
   Column stiffener to flange weld                                     | 0.500 in
   Column stiffener to web weld                                        | 0.250 in
-----------------------------------------------------------------------+---------------------
Design calculations                                                    |
```

```
----------------------------------------------------------------------+--------------------
                                                                      |
Connection design forces:                                             |
   Required plastic moment capacity of beam                           |
      [Mpe=1.1*Ry*Fyb*Zx]                                             | 2809.620 kip in
   Distance from the face of the column to the plastic hinge (Lp)     | 6.900 in
   Required moment capacity of connection                             |
      [Muc=Mpe+V*Lp]                                                  | 3051.120 kip in
   Flange force                                                       |
      [Fu=Muc/(d-tf)]                                                 | 227.441 kip
                                                                      |
Bolt Tension check:                                                   |
   Distance of the centerline of the beam compression flange to the   |
      1st tension bolt row (h0)                                       | 15.357 in
      2nd tension bolt row (h1)                                       | 11.665 in
   Bolt nominal tensile stress                                        | 90.000 ksi
   Required bolt diameter                                             |
      [db'=sqrt(2*Muc/(PI*phi_bv*Fnt*(h0+h1)))]                       | 1.032 in
   Interaction ratio in bolt tension                                  |
      [db'/db]                                                        | 0.917
                                                                      |
Bolt Shear Rupture Check:                                             |
   Number of bolts provided at compression flange (nc)                | 4.000
   Nominal shear strength of bolt (Vb)                                | 53.684 kip
   Nominal shear strength of connectionType                           |
      [Rn=nc*Vb]                                                      | 214.736 kip
   LRFD factor in bolt shear (phi)                                    | 0.750
   Bolt shear capacity                                                |
      [Ra=Rn*phi]                                                     | 161.052 kip
   Interaction ratio in bolt shear                                    |
      [Vu/Ra]                                                         | 0.217
                                                                      |
Bolt Bearing on Plate:                                                |
   Nominal strength in bolt bearing on plate (Rn)                     | 557.670 kip
   LRFD factor in bolt bearing (phi)                                  | 0.750
   Bolt bearing on plate capacity                                     |
      [Ra=Rn*phi]                                                     | 418.252 kip
   Interaction ratio in bolt bearing on plate                         |
      [Vu/Ra]                                                         | 0.084
                                                                      |
Bolt Bearing on Column Flange:                                        |
   Nominal strength in bolt bearing on flange (Rn)                    | 398.482 kip
   LRFD factor in bolt bearing (phi)                                  | 0.750
   Bolt bearing on flange capacity                                    |
      [Ra=Rn*phi]                                                     | 298.861 kip
   Interaction ratio in bolt bearing on flange                        |
      [Vu/Ra]                                                         | 0.117
                                                                      |
Plate Thickness Calculation:                                          |
   Bolt tensile strength (Pt)                                         | 89.416 kip
   No prying design moment                                            |
      [Mnp= 2*Pt*(h0+h1)]                                             | 4832.510 kip in
   LRFD factor in bolt rupture (phi)                                  | 0.750
```

```
   LRFD factor in bending (phi_b)                              | 0.900
   Yield line parameter for plate (Yp)                         | 131.532 in
   Required minimum plate thickness                            |
      [tp‘=sqrt(1.11*phi*Mnp/(phi_b*Fyp*Yp))]                  | 0.972 in
   Interaction ratio for plate thickness                       |
      [tp‘/tp]                                                 | 0.972
                                                               |
Plate Shear Yielding Check:                                    |
   Gross shear area of plate                                   | 12.000 in^2
   Nominal strength in shear yielding (Rn)                     | 259.200 kip
   LRFD factor in shear yielding (phi)                         | 1.000
   Shear yielding capacity of plate                            |
      [Ra=Rn*phi]                                              | 259.200 kip
   Interaction ratio in shear yielding                         |
      [Fu/(2*Ra)]                                              | 0.439
                                                               |
Plate Shear Rupture Check:                                     |
   Net shear area of plate                                     | 9.625 in^2
   Nominal strength in shear rupture (Rn)                      | 334.950 kip
   LRFD factor in shear rupture (phi)                          | 0.750
   Shear rupture capacity of plate                             |
      [Ra=Rn*phi]                                              | 251.212 kip
   Interaction ratio in shear rupture                          |
      [Fu/(2*Ra)]                                              | 0.453
                                                               |
Beam Web to End Plate Weld Tension Check:                      |
   Required tension strength of weld at beam web (fw)          | 9.720 kip/in
   Nominal strength of weld at beam web (fn)                   | 22.270 kip/in
   LRFD factor for weld (phi)                                  | 0.750
   Beam web weld capacity                                      |
      [fa=fn*phi]                                              | 16.703 kip/in
   Interaction ratio for beam web weld in tension              |
      [fw/fa]                                                  | 0.582
                                                               |
Beam Web to End Plate Weld Shear Check:                        |
   Total weld length                                           | 9.760 in
   Nominal strength of weld at beam web (Rn)                   | 72.453 kip
   LRFD factor for weld (phi)                                  | 0.750
   Beam web weld capacity                                      |
      [Ra=Rn*phi]                                              | 54.340 kip
   Interaction ratio for beam web weld in shear                |
      [Vu/Ra]                                                  | 0.644
                                                               |
Column Flange Yielding Check:                                  |
   Column yield line parameter (Yc)                            | 185.904 in
   LRFD factor in bolt rupture (phi)                           | 0.750
   LRFD factor in flexure (phi_b)                              | 0.900
   Required minimum column flange thickness                    |
      [tf‘=sqrt(1.11*phi*Mnp/(phi_b*Fyc*Yc))]                  | 0.817 in
   Interaction ratio for column flange yielding                |
      [ratio=tf‘/tf]                                           | 0.830
                                                               |
Column Web Yielding Check:                                     |
```

```
   Bearing length                                                          |
      [N=tf+2*wb]                                                          | 0.705 in
   Nominal strength of column web in local yielding (Rn)                   | 123.727 kip
   LRFD factor in local web yielding (phi)                                 | 1.000
   Column local web yielding capacity                                      |
      [Ra=Rn*phi]                                                          | 123.727 kip
   Interaction ratio in web local yielding                                 |
      [Fu/Ra]                                                              | 0.000
                                                                           |
Column Local Web Crippling Check:                                          |
   Nominal strength in column web crippling (Rn)                           | 191.829 kip
   LRFD factor in web crippling (phi)                                      | 0.750
   Column web crippling capacity                                           |
      [Ra=Rn*phi]                                                          | 143.872 kip
   Interaction ratio in web crippling                                      |
      [Fu/Ra]                                                              | 0.000
                                                                           |
Column Panel Shear Check:                                                  |
   Nominal strength of column in panel shear                               | 262.193 kip
   LRFD factor for column panel shear (phi)                                | 0.900
   Column panel shear capacity                                             |
      [Ra=Rn*phi]                                                          | 235.973 kip
   Interaction ratio for column panel shear                                |
      [Fu/Ra]                                                              | 0.964
                                                                           |
Column Web Shear Buckling Check:                                           |
   Minimum thickness of web to prevent shear buckling (tw‘)                | 0.316 in
   Interaction ratio for thickness of web                                  |
      [tw‘/tw]                                                             | 0.540
                                                                           |
Transverse Stiffener Axial Strength:                                       |
   Required strength of column stiffener (Ru)                              | 103.713 kip
   Nominal yield strength of stiffener (Rn)                                | 108.000 kip
   LRFD factor in axial strength (phi)                                     | 0.900
   Stiffener axial yield capacity                                          |
      [Ra=Rn*phi]                                                          | 97.200 kip
   Interaction ratio for stiffener in axial strength                       |
      [Ru/Ra]                                                              | 0.534
                                                                           |
Transverse stiffener shear strength:                                       |
   Nominal shear strength of stiffener (Rn)                                | 281.448 kip
   LRFD factor in stiffener shear strength (phi)                           | 0.900
   Stiffener shear capacity                                                |
      [Ra=Rn*phi]                                                          | 253.303 kip
   Interaction ratio for stiffener in shear                                |
      [Ru/(2*Ra)]                                                          | 0.205
                                                                           |
Transverse Stiffener Thickness Check:                                      |
   Minimum required thickness of stiffener (tsc‘)                          | 0.253 in
   Interaction ratio for stiffener thickness                               |
      [tsc‘/tsc]                                                           | 0.253
                                                                           |
Transverse Stiffener to Flange Weld Check:                                 |
```

```
   Required strength for stifferner to flange weld (Ps)               | 32.400 kip/in
   Nominal strength of stiffener to flange weld (Rn)                  | 44.541 kip/in
   LRFD factor for weld strength (phi)                                | 0.750
   Stiffener to flange weld capacity                                  |
      [Ra=Rn*phi]                                                     | 33.406 kip
   Interaction ratio for stiffener to flange weld                     |
      [Ps/Ra]                                                         | 0.970
                                                                      |
Transverse Stiffener to Web Weld Check:                               |
   Nominal strength of stiffener to web weld (Rn)                     | 193.456 kip/in
   LRFD factor for weld strength (phi)                                | 0.750
   Stiffener to web weld capacity                                     |
      [Ra=Rn*phi]                                                     | 145.092 kip/in
   Interaction ratio for stiffener to web weld                        |
      [Ru/(2*Ra)]                                                     | 0.357
```

3.2 Validation problem 2

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_2
----------------------------------------------------------------------+---------------------
Design Summary                                                        |
----------------------------------------------------------------------+---------------------
   Connection is OK                                                   |
   Maximum interaction ratio                                          | 0.997
----------------------------------------------------------------------+---------------------
Design Inputs                                                         |
----------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                     | 4 BOLT - STIFFENED
   Design method                                                      | LRFD
   Shear force in connection (V)                                      | 45.000 kip
   Column axial force                                                 | 350.000 kip
   Modulus of elasticity of steel                                     | 29000.000 ksi
   Yield strength of beam (Fyb)                                       | 50.000 ksi
   Tensile strength of beam                                           | 65.000 ksi
   Yield strength of column                                           | 50.000 ksi
   Tensile strength of column                                         | 65.000 ksi
   Yield strength of plate                                            | 36.000 ksi
   Tensile strength of plate                                          | 58.000 ksi
   Tensile strength of weld                                           | 70.000 ksi
   Material overstrength factor for beam (Ry)                         | 1.100
   End plate dimensions (b x d x tp)                                  | 12 x 23 x 1 in
   Bolt diamter                                                       | 1.125 in
   Bolt grade                                                         | ASTM A325
   Thickness of beam web to plate fillet weld                         | 0.250 in
   Thickness of flange weld reinforing fillet weld                    | 0.160 in
   Thickness of end plate stiffener (tsp)                             | 0.500 in
   Length of end plate stiffener                                      | 8.000 in
   Thickness of plate stiffener to beam flange weld                   | 0.250 in
   Beam section property                                              | W14X34
      Depth (d)                                                       | 14.000 in
      Flange width                                                    | 6.750 in
```

```
      Web thickness                                                | 0.285 in
      Flange thickness (tf)                                        | 0.455 in
      Fillet dimension                                             | 0.855 in
      Plastic section modulus (Zx)                                 | 54.600 in^3
   Column section property                                         | W14X120
      Depth                                                        | 14.500 in
      Flange width                                                 | 14.700 in
      Web thickness                                                | 0.590 in
      Flange thickness                                             | 0.940 in
      Fillet dimension                                             | 1.540 in
   Thickness of column stiffener (tsc)                             | 0.750 in
   Width of column stiffener                                       | 4.000 in
   Column stiffener clip dimension                                 | 1.000 in
   Column stiffener to flange weld                                 | 0.500 in
   Column stiffener to web weld                                    | 0.250 in
-------------------------------------------------------------------+--------------------
Design calculations                                                |
-------------------------------------------------------------------+--------------------
                                                                   |
Connection design forces:                                          |
   Required plastic moment capacity of beam                        |
      [Mpe=1.1*Ry*Fyb*Zx]                                          | 3303.300 kip in
   Distance from the face of the column to the plastic hinge (Lp)  | 9.000 in
   Required moment capacity of connection                          |
      [Muc=Mpe+V*Lp]                                               | 3708.300 kip in
   Flange force                                                    |
      [Fu=Muc/(d-tf)]                                              | 273.776 kip
                                                                   |
Bolt Tension check:                                                |
   Distance of the centerline of the beam compression flange to the |
      1st tension bolt row (h0)                                    | 16.023 in
      2nd tension bolt row (h1)                                    | 11.795 in
   Bolt nominal tensile stress                                     | 90.000 ksi
   Required bolt diameter                                          |
      [db‘=sqrt(2*Muc/(PI*phi_bv*Fnt*(h0+h1)))]                    | 1.121 in
   Interaction ratio in bolt tension                               |
      [db‘/db]                                                     | 0.997
                                                                   |
Bolt Shear Rupture Check:                                          |
   Number of bolts provided at compression flange (nc)             | 4.000
   Nominal shear strength of bolt (Vb)                             | 53.684 kip
   Nominal shear strength of connectionType                        |
      [Rn=nc*Vb]                                                   | 214.736 kip
   LRFD factor in bolt shear (phi)                                 | 0.750
   Bolt shear capacity                                             |
      [Ra=Rn*phi]                                                  | 161.052 kip
   Interaction ratio in bolt shear                                 |
      [Vu/Ra]                                                      | 0.279
                                                                   |
Bolt Bearing on Plate:                                             |
   Nominal strength in bolt bearing on plate (Rn)                  | 543.750 kip
   LRFD factor in bolt bearing (phi)                               | 0.750
   Bolt bearing on plate capacity                                  |
```

```
      [Ra=Rn*phi]                                                    | 407.812 kip
   Interaction ratio in bolt bearing on plate                        |
      [Vu/Ra]                                                        | 0.110
                                                                     |
Bolt Bearing on Column Flange:                                       |
   Nominal strength in bolt bearing on flange (Rn)                   | 352.853 kip
   LRFD factor in bolt bearing (phi)                                 | 0.750
   Bolt bearing on flange capacity                                   |
      [Ra=Rn*phi]                                                    | 264.639 kip
   Interaction ratio in bolt bearing on flange                       |
      [Vu/Ra]                                                        | 0.170
                                                                     |
Plate Thickness Calculation:                                         |
   Bolt tensile strength (Pt)                                        | 89.416 kip
   No prying design moment                                           |
      [Mnp= 2*Pt*(h0+h1)]                                            | 4974.682 kip in
   LRFD factor in bolt rupture (phi)                                 | 0.750
   LRFD factor in bending (phi_b)                                    | 0.900
   Yield line parameter for plate (Yp)                               | 163.569 in
   Required minimum plate thickness                                  |
      [tp`=sqrt(1.11*phi*Mnp/(phi_b*Fyp*Yp))]                        | 0.884 in
   Interaction ratio for plate thickness                             |
      [tp`/tp]                                                       | 0.884
                                                                     |
Beam Web to End Plate Weld Tension Check:                            |
   Required tension strength of weld at beam web (fw)                | 14.250 kip/in
   Nominal strength of weld at beam web (fn)                         | 22.270 kip/in
   LRFD factor for weld (phi)                                        | 0.750
   Beam web weld capacity                                            |
      [fa=fn*phi]                                                    | 16.703 kip/in
   Interaction ratio for beam web weld in tension                    |
      [fw/fa]                                                        | 0.853
                                                                     |
Beam Web to End Plate Weld Shear Check:                              |
   Total weld length                                                 | 9.880 in
   Nominal strength of weld at beam web (Rn)                         | 73.344 kip
   LRFD factor for weld (phi)                                        | 0.750
   Beam web weld capacity                                            |
      [Ra=Rn*phi]                                                    | 55.008 kip
   Interaction ratio for beam web weld in shear                      |
      [Vu/Ra]                                                        | 0.818
                                                                     |
End Plate Stiffener Thickness Check:                                 |
   Required minimum stiffener thickness (tsp`)                       | 0.396 in
   Interaction ratio for stiffener thickness                         |
      [tsp`/tsp]                                                     | 0.792
                                                                     |
End Plate Stiffener to Flange Weld Check:                            |
   Required strength of stiffener weld                               |
      [Vs=0.6*Fyp*ts]                                                | 10.800 kip/in
   Nominal strength of stiffener to flange weld (Rn)                 | 14.847 kip/in
   LRFD factor for weld (phi)                                        | 0.750
   Stiffener to flange weld capacity                                 |
```

```
      [Ra=Rn*phi]                                                       | 11.135 kip
   Interaction ratio for stiffener to flange weld                       |
      [Vs/Ra]                                                           | 0.970
                                                                        |
Column Flange Yielding Check:                                           |
   Column yield line parameter (Yc)                                     | 220.189 in
   LRFD factor in bolt rupture (phi)                                    | 0.750
   LRFD factor in flexure (phi_b)                                       | 0.900
   Required minimum column flange thickness                             |
      [tf‘=sqrt(1.11*phi*Mnp/(phi_b*Fyc*Yc))]                           | 0.647 in
   Interaction ratio for column flange yielding                         |
      [ratio=tf‘/tf]                                                    | 0.688
                                                                        |
Column Web Yielding Check:                                              |
   Bearing length                                                       |
      [N=tf+2*wb]                                                       | 0.775 in
   Nominal strength of column web in local yielding (Rn)                | 188.653 kip
   LRFD factor in local web yielding (phi)                              | 1.000
   Column local web yielding capacity                                   |
      [Ra=Rn*phi]                                                       | 188.653 kip
   Interaction ratio in web local yielding                              |
      [Fu/Ra]                                                           | 0.000
                                                                        |
Column Local Web Crippling Check:                                       |
   Nominal strength in column web crippling (Rn)                        | 228.509 kip
   LRFD factor in web crippling (phi)                                   | 0.750
   Column web crippling capacity                                        |
      [Ra=Rn*phi]                                                       | 171.381 kip
   Interaction ratio in web crippling                                   |
      [Fu/Ra]                                                           | 0.000
                                                                        |
Column Panel Shear Check:                                               |
   Nominal strength of column in panel shear                            | 340.150 kip
   LRFD factor for column panel shear (phi)                             | 0.900
   Column panel shear capacity                                          |
      [Ra=Rn*phi]                                                       | 306.135 kip
   Interaction ratio for column panel shear                             |
      [Fu/Ra]                                                           | 0.894
                                                                        |
Column Web Shear Buckling Check:                                        |
   Minimum thickness of web to prevent shear buckling (tw‘)             | 0.291 in
   Interaction ratio for thickness of web                               |
      [tw‘/tw]                                                          | 0.493
                                                                        |
Transverse Stiffener Axial Strength:                                    |
   Required strength of column stiffener (Ru)                           | 102.395 kip
   Nominal yield strength of stiffener (Rn)                             | 81.000 kip
   LRFD factor in axial strength (phi)                                  | 0.900
   Stiffener axial yield capacity                                       |
      [Ra=Rn*phi]                                                       | 72.900 kip
   Interaction ratio for stiffener in axial strength                    |
      [Ru/Ra]                                                           | 0.702
                                                                        |
```

```
Transverse stiffener shear strength:                                    |
   Nominal shear strength of stiffener (Rn)                             | 172.044 kip
   LRFD factor in stiffener shear strength (phi)                        | 0.900
   Stiffener shear capacity                                             |
      [Ra=Rn*phi]                                                       | 154.840 kip
   Interaction ratio for stiffener in shear                             |
      [Ru/(2*Ra)]                                                       | 0.331
                                                                        |
Transverse Stiffener Thickness Check:                                   |
   Minimum required thickness of stiffener (tsc‘)                       | 0.253 in
   Interaction ratio for stiffener thickness                            |
      [tsc‘/tsc]                                                        | 0.337
                                                                        |
Transverse Stiffener to Flange Weld Check:                              |
   Required strength for stifferner to flange weld (Ps)                 | 24.300 kip/in
   Nominal strength of stiffener to flange weld (Rn)                    | 44.541 kip/in
   LRFD factor for weld strength (phi)                                  | 0.750
   Stiffener to flange weld capacity                                    |
      [Ra=Rn*phi]                                                       | 33.406 kip
   Interaction ratio for stiffener to flange weld                       |
      [Ps/Ra]                                                           | 0.727
                                                                        |
Transverse Stiffener to Web Weld Check:                                 |
   Nominal strength of stiffener to web weld (Rn)                       | 157.675 kip/in
   LRFD factor for weld strength (phi)                                  | 0.750
   Stiffener to web weld capacity                                       |
      [Ra=Rn*phi]                                                       | 118.256 kip/in
   Interaction ratio for stiffener to web weld                          |
      [Ru/(2*Ra)]                                                       | 0.433
```

3.3 Validation problem 3

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_3
------------------------------------------------------------------------+---------------------
Design Summary                                                          |
------------------------------------------------------------------------+---------------------
   Connection is OK                                                     |
   Maximum interaction ratio                                            | 0.970
------------------------------------------------------------------------+---------------------
Design Inputs                                                           |
------------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                       | 8 BOLT - STIFFENED
   Design method                                                        | LRFD
   Shear force in connection (V)                                        | 60.000 kip
   Column axial force                                                   | 590.000 kip
   Modulus of elasticity of steel                                       | 29000.000 ksi
   Yield strength of beam (Fyb)                                         | 50.000 ksi
   Tensile strength of beam                                             | 65.000 ksi
   Yield strength of column                                             | 50.000 ksi
   Tensile strength of column                                           | 65.000 ksi
   Yield strength of plate                                              | 36.000 ksi
```

```
   Tensile strength of plate                                             | 58.000 ksi
   Tensile strength of weld                                              | 70.000 ksi
   Material overstrength factor for beam (Ry)                            | 1.100
   End plate dimensions (b x d x tp)                                     | 12 x 30 x 1.25 in
   Bolt diamter                                                          | 1.125 in
   Bolt grade                                                            | ASTM A490
   Thickness of beam web to plate fillet weld                            | 0.313 in
   Thickness of flange weld reinforing fillet weld                       | 0.160 in
   Thickness of end plate stiffener (tsp)                                | 0.500 in
   Length of end plate stiffener                                         | 13.000 in
   Thickness of plate stiffener to beam flange weld                      | 0.250 in
   Beam section property                                                 | W16X45
      Depth (d)                                                          | 16.100 in
      Flange width                                                       | 7.040 in
      Web thickness                                                      | 0.345 in
      Flange thickness (tf)                                              | 0.565 in
      Fillet dimension                                                   | 0.967 in
      Plastic section modulus (Zx)                                       | 82.300 in^3
   Column section property                                               | W14X159
      Depth                                                              | 15.000 in
      Flange width                                                       | 15.600 in
      Web thickness                                                      | 0.745 in
      Flange thickness                                                   | 1.190 in
      Fillet dimension                                                   | 1.790 in
   Thickness of column stiffener (tsc)                                   | 0.750 in
   Width of column stiffener                                             | 6.000 in
   Column stiffener clip dimension                                       | 1.000 in
   Column stiffener to flange weld                                       | 0.500 in
   Column stiffener to web weld                                          | 0.250 in
-----------------------------------------------------------------------+---------------------
Design calculations                                                    |
-----------------------------------------------------------------------+---------------------
                                                                       |
Connection design forces:                                              |
   Required plastic moment capacity of beam                            |
      [Mpe=1.1*Ry*Fyb*Zx]                                              | 4979.150 kip in
   Distance from the face of the column to the plastic hinge (Lp)      | 14.250 in
   Required moment capacity of connection                              |
      [Muc=Mpe+V*Lp]                                                   | 5834.150 kip in
   Flange force                                                        |
      [Fu=Muc/(d-tf)]                                                  | 375.549 kip
                                                                       |
Bolt Tension check:                                                    |
   Distance of the centerline of the beam compression flange to the    |
      1st tension bolt row (h1)                                        | 21.068 in
      2nd tension bolt row (h2)                                        | 17.568 in
      3rd tension bolt row (h3)                                        | 13.785 in
      4th tension bolt row (h4)                                        | 10.285 in
   Bolt nominal tensile stress                                         | 113.000 ksi
   Required bolt diameter                                              |
      [db'=sqrt(2*Muc/(PI*phi_bv*Fnt*(h1+h2+h3+h4)))]                  | 0.836 in
   Interaction ratio in bolt tension                                   |
      [db'/db]                                                         | 0.743
```

```
                                                                       |
Bolt Shear Rupture Check:                                              |
   Number of bolts provided at compression flange (nc)                 | 8.000
   Nominal shear strength of bolt (Vb)                                 | 67.602 kip
   Nominal shear strength of connectionType                            |
      [Rn=nc*Vb]                                                       | 540.817 kip
   LRFD factor in bolt shear (phi)                                     | 0.750
   Bolt shear capacity                                                 |
      [Ra=Rn*phi]                                                      | 405.613 kip
   Interaction ratio in bolt shear                                     |
      [Vu/Ra]                                                          | 0.148
                                                                       |
Bolt Bearing on Plate:                                                 |
   Nominal strength in bolt bearing on plate (Rn)                      | 1366.987 kip
   LRFD factor in bolt bearing (phi)                                   | 0.750
   Bolt bearing on plate capacity                                      |
      [Ra=Rn*phi]                                                      | 1025.241 kip
   Interaction ratio in bolt bearing on plate                          |
      [Vu/Ra]                                                          | 0.059
                                                                       |
Bolt Bearing on Column Flange:                                         |
   Nominal strength in bolt bearing on flange (Rn)                     | 1670.760 kip
   LRFD factor in bolt bearing (phi)                                   | 0.750
   Bolt bearing on flange capacity                                     |
      [Ra=Rn*phi]                                                      | 1253.070 kip
   Interaction ratio in bolt bearing on flange                         |
      [Vu/Ra]                                                          | 0.048
                                                                       |
Plate Thickness Calculation:                                           |
   Bolt tensile strength (Pt)                                          | 112.267 kip
   No prying design moment                                             |
      [Mnp= 2*Pt*(h1+h2+h3+h4)]                                        | 14079.438 kip in
   LRFD factor in bolt rupture (phi)                                   | 0.750
   LRFD factor in bending (phi_b)                                      | 0.900
   Yield line parameter for plate (Yp)                                 | 248.609 in
   Required minimum plate thickness                                    |
      [tp‘=sqrt(1.11*phi*Mnp/(phi_b*Fyp*Yp))]                          | 1.206 in
   Interaction ratio for plate thickness                               |
      [tp‘/tp]                                                         | 0.965
                                                                       |
Beam Web to End Plate Weld Tension Check:                              |
   Required tension strength of weld at beam web (fw)                  | 17.250 kip/in
   Nominal strength of weld at beam web (fn)                           | 27.883 kip/in
   LRFD factor for weld (phi)                                          | 0.750
   Beam web weld capacity                                              |
      [fa=fn*phi]                                                      | 20.912 kip/in
   Interaction ratio for beam web weld in tension                      |
      [fw/fa]                                                          | 0.825
                                                                       |
Beam Web to End Plate Weld Shear Check:                                |
   Total weld length                                                   | 13.636 in
   Nominal strength of weld at beam web (Rn)                           | 126.736 kip
   LRFD factor for weld (phi)                                          | 0.750
```

```
   Beam web weld capacity                                                 |
      [Ra=Rn*phi]                                                         | 95.052 kip
   Interaction ratio for beam web weld in shear                           |
      [Vu/Ra]                                                             | 0.631
                                                                          |
End Plate Stiffener Thickness Check:                                      |
   Required minimum stiffener thickness (tsp‘)                            | 0.479 in
   Interaction ratio for stiffener thickness                              |
      [tsp‘/tsp]                                                          | 0.958
                                                                          |
End Plate Stiffener to Flange Weld Check:                                 |
   Required strength of stiffener weld                                    |
      [Vs=0.6*Fyp*ts]                                                     | 10.800 kip/in
   Nominal strength of stiffener to flange weld (Rn)                      | 14.847 kip/in
   LRFD factor for weld (phi)                                             | 0.750
   Stiffener to flange weld capacity                                      |
      [Ra=Rn*phi]                                                         | 11.135 kip
   Interaction ratio for stiffener to flange weld                         |
      [Vs/Ra]                                                             | 0.970
                                                                          |
Column Flange Yielding Check:                                             |
   Column yield line parameter (Yc)                                       | 310.592 in
   LRFD factor in bolt rupture (phi)                                      | 0.750
   LRFD factor in flexure (phi_b)                                         | 0.900
   Required minimum column flange thickness                               |
      [tf‘=sqrt(1.11*phi*Mnp/(phi_b*Fyc*Yc))]                             | 0.916 in
   Interaction ratio for column flange yielding                           |
      [ratio=tf‘/tf]                                                      | 0.770
                                                                          |
Column Web Yielding Check:                                                |
   Bearing length                                                         |
      [N=tf+2*wb]                                                         | 0.885 in
   Nominal strength of column web in local yielding (Rn)                  | 279.561 kip
   LRFD factor in local web yielding (phi)                                | 1.000
   Column local web yielding capacity                                     |
      [Ra=Rn*phi]                                                         | 279.561 kip
   Interaction ratio in web local yielding                                |
      [Fu/Ra]                                                             | 0.000
                                                                          |
Column Local Web Crippling Check:                                         |
   Nominal strength in column web crippling (Rn)                          | 734.991 kip
   LRFD factor in web crippling (phi)                                     | 0.750
   Column web crippling capacity                                          |
      [Ra=Rn*phi]                                                         | 551.243 kip
   Interaction ratio in web crippling                                     |
      [Fu/Ra]                                                             | 0.000
                                                                          |
Column Panel Shear Check:                                                 |
   Nominal strength of column in panel shear                              | 458.741 kip
   LRFD factor for column panel shear (phi)                               | 0.900
   Column panel shear capacity                                            |
      [Ra=Rn*phi]                                                         | 412.867 kip
   Interaction ratio for column panel shear                               |
```

```
      [Fu/Ra]                                                       | 0.910
                                                                    |
Column Web Shear Buckling Check:                                    |
   Minimum thickness of web to prevent shear buckling (tw‘)         | 0.313 in
   Interaction ratio for thickness of web                           |
      [tw‘/tw]                                                      | 0.420
                                                                    |
Transverse Stiffener Axial Strength:                                |
   Required strength of column stiffener (Ru)                       | 95.988 kip
   Nominal yield strength of stiffener (Rn)                         | 135.000 kip
   LRFD factor in axial strength (phi)                              | 0.900
   Stiffener axial yield capacity                                   |
      [Ra=Rn*phi]                                                   | 121.500 kip
   Interaction ratio for stiffener in axial strength                |
      [Ru/Ra]                                                       | 0.395
                                                                    |
Transverse stiffener shear strength:                                |
   Nominal shear strength of stiffener (Rn)                         | 172.044 kip
   LRFD factor in stiffener shear strength (phi)                    | 0.900
   Stiffener shear capacity                                         |
      [Ra=Rn*phi]                                                   | 154.840 kip
   Interaction ratio for stiffener in shear                         |
      [Ru/(2*Ra)]                                                   | 0.310
                                                                    |
Transverse Stiffener Thickness Check:                               |
   Minimum required thickness of stiffener (tsc‘)                   | 0.379 in
   Interaction ratio for stiffener thickness                        |
      [tsc‘/tsc]                                                    | 0.505
                                                                    |
Transverse Stiffener to Flange Weld Check:                          |
   Required strength for stifferner to flange weld (Ps)             | 24.300 kip/in
   Nominal strength of stiffener to flange weld (Rn)                | 44.541 kip/in
   LRFD factor for weld strength (phi)                              | 0.750
   Stiffener to flange weld capacity                                |
      [Ra=Rn*phi]                                                   | 33.406 kip
   Interaction ratio for stiffener to flange weld                   |
      [Ps/Ra]                                                       | 0.727
                                                                    |
Transverse Stiffener to Web Weld Check:                             |
   Nominal strength of stiffener to web weld (Rn)                   | 157.675 kip/in
   LRFD factor for weld strength (phi)                              | 0.750
   Stiffener to web weld capacity                                   |
      [Ra=Rn*phi]                                                   | 118.256 kip/in
   Interaction ratio for stiffener to web weld                      |
      [Ru/(2*Ra)]                                                   | 0.406
```

3.4 Validation problem 4

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_4
--------------------------------------------------------------------+--------------------
Design Summary                                                      |
```

```
-----------------------------------------------------------------------+---------------------
   Connection is OK                                                    |
   Maximum interaction ratio                                           | 0.988
-----------------------------------------------------------------------+---------------------
Design Inputs                                                          |
-----------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                      | 4 BOLT - UNSTIFFENED
   Design method                                                       | ASD
   Shear force in connection (V)                                       | 36000.000 N
   Column axial force                                                  | 455000.000 N
   Modulus of elasticity of steel                                      | 200000.000 MPa
   Yield strength of beam (Fyb)                                        | 250.000 MPa
   Tensile strength of beam                                            | 400.000 MPa
   Yield strength of column                                            | 250.000 MPa
   Tensile strength of column                                          | 400.000 MPa
   Yield strength of plate                                             | 250.000 MPa
   Tensile strength of plate                                           | 400.000 MPa
   Tensile strength of weld                                            | 482.000 MPa
   Material overstrength factor for beam (Ry)                          | 1.500
   End plate dimensions (b x d x tp)                                   | 400 x 650 x 30 mm
   Bolt diamter                                                        | 27.000 mm
   Bolt grade                                                          | ASTM A490
   Thickness of beam web to plate fillet weld                          | 6.000 mm
   Thickness of flange weld reinforing fillet weld                     | 4.000 mm
   Beam section property                                               | W360X44
      Depth (d)                                                        | 351.000 mm
      Flange width                                                     | 171.000 mm
      Web thickness                                                    | 6.860 mm
      Flange thickness (tf)                                            | 9.780 mm
      Fillet dimension                                                 | 19.900 mm
      Plastic section modulus (Zx)                                     | 775000.000 mm^3
   Column section property                                             | W360X347
      Depth                                                            | 406.000 mm
      Flange width                                                     | 404.000 mm
      Web thickness                                                    | 27.200 mm
      Flange thickness                                                 | 43.700 mm
      Fillet dimension                                                 | 58.900 mm
-----------------------------------------------------------------------+---------------------
Design calculations                                                    |
-----------------------------------------------------------------------+---------------------
                                                                       |
Connection design forces:                                              |
   Required plastic moment capacity of beam                            |
      [Mpe=1.1*Ry*Fyb*Zx]                                              | 319687500.000 N mm
   Distance from the face of the column to the plastic hinge (Lp)      | 175.500 mm
   Required moment capacity of connection                              |
      [Muc=Mpe+1.5*V*Lp]                                               | 326005500.000 N mm
   Flange force                                                        |
      [Fu=Muc/(d-tf)]                                                  | 955411.465 N
                                                                       |
Bolt Tension check:                                                    |
   Distance of the centerline of the beam compression flange to the    |
      1st tension bolt row (h0)                                        | 446.110 mm
```

```
      2nd tension bolt row (h1)                                   | 301.220 mm
   Bolt nominal tensile stress                                    | 780.000 MPa
   Required bolt diameter                                         |
      [db'=sqrt(2*Muc/(PI*phi_bv*Fnt*(h0+h1)))]                   | 26.685 mm
   Interaction ratio in bolt tension                              |
      [db'/db]                                                    | 0.988
                                                                  |
Bolt Shear Rupture Check:                                         |
   Number of bolts provided at compression flange (nc)            | 4.000
   Nominal shear strength of bolt (Vb)                            | 268563.236 N
   Nominal shear strength of connectionType                       |
      [Rn=nc*Vb]                                                  | 1074252.942 N
   ASD factor in bolt shear (omega)                               | 2.000
   Bolt shear capacity                                            |
      [Ra=Rn/omega]                                               | 537126.471 N
   Interaction ratio in bolt shear                                |
      [Vu/Ra]                                                     | 0.067
                                                                  |
Bolt Bearing on Plate:                                            |
   Nominal strength in bolt bearing on plate (Rn)                 | 2548800.000 N
   ASD factor in bolt bearing (omega)                             | 2.000
   Bolt bearing on plate capacity                                 |
      [Ra=Rn/omega]                                               | 1274400.000 N
   Interaction ratio in bolt bearing on plate                     |
      [Vu/Ra]                                                     | 0.028
                                                                  |
Bolt Bearing on Column Flange:                                    |
   Nominal strength in bolt bearing on flange (Rn)                | 1636128.000 N
   ASD factor in bolt bearing (omega)                             | 2.000
   Bolt bearing on flange capacity                                |
      [Ra=Rn/omega]                                               | 818064.000 N
   Interaction ratio in bolt bearing on flange                    |
      [Vu/Ra]                                                     | 0.044
                                                                  |
Plate Thickness Calculation:                                      |
   Bolt tensile strength (Pt)                                     | 446366.700 N
   No prying design moment                                        |
      [Mnp= 2*Pt*(h0+h1)]                                         | 667166451.822 N mm
   ASD factor in bolt rupture (omega)                             | 2.000
   ASD factor in bending (omega_b)                                | 1.670
   Yield line parameter for plate (Yp)                            | 3442.751 mm
   Required minimum plate thickness                               |
      [tp'=sqrt(1.11*omega_b*Mnp/(omega*Fyp*Yp))]                 | 26.804 mm
   Interaction ratio for plate thickness                          |
      [tp'/tp]                                                    | 0.893
                                                                  |
Plate Shear Yielding Check:                                       |
   Gross shear area of plate                                      | 12000.000 mm^2
   Nominal strength in shear yielding (Rn)                        | 1800000.000 N
   ASD factor in shear yielding (omega)                           | 1.500
   Shear yielding capacity of plate                               |
      [Ra=Rn/omega]                                               | 1200000.000 N
   Interaction ratio in shear yielding                            |
```

```
      [Fu/(2*Ra)]                                            | 0.398
                                                             |
Plate Shear Rupture Check:                                   |
   Net shear area of plate                                   | 10200.000 mm^2
   Nominal strength in shear rupture (Rn)                    | 2448000.000 N
   ASD factor in shear rupture (omega)                       | 2.000
   Shear rupture capacity of plate                           |
      [Ra=Rn/omega]                                          | 1224000.000 N
   Interaction ratio in shear rupture                        |
      [Fu/(2*Ra)]                                            | 0.390
                                                             |
Beam Web to End Plate Weld Tension Check:                    |
   Required tension strength of weld at beam web (fw)        | 1715.000 N/mm
   Nominal strength of weld at beam web (fn)                 | 3680.359 N/mm
   ASD factor in weld (omega)                                | 2.000
   Beam web weld capacity                                    |
      [fa=fn/omega]                                          | 1840.180 N/mm
   Interaction ratio for beam web weld in tension            |
      [fw/fa]                                                | 0.932
                                                             |
Beam Web to End Plate Weld Shear Check:                      |
   Total weld length                                         | 257.840 mm
   Nominal strength of weld at beam web (Rn)                 | 316314.605 N
   ASD factor in weld (omega)                                | 2.000
   Beam web weld capacity                                    |
      [Ra=Rn/omega]                                          | 158157.303 N
   Interaction ratio for beam web weld in shear              |
      [Vu/Ra]                                                | 0.228
                                                             |
Column Flange Yielding Check:                                |
   Column yield line parameter (Yc)                          | 3351.399 mm
   ASD factor in bolt rupture (omega)                        | 2.000
   ASD factor in flexure (omega_b)                           | 1.670
   Required minimum column flange thickness                  |
      [tf'=sqrt(1.11*omega_b*Mnp/(omega*Fyc*Yc))]            | 27.167 mm
   Interaction ratio for column flange yielding              |
      [ratio=tf'/tf]                                         | 0.622
                                                             |
Column Web Yielding Check:                                   |
   Bearing length                                            |
      [N=tf+2*wb]                                            | 17.780 mm
   Nominal strength of column web in local yielding (Rn)     | 1526464.000 N
   LRFD factor in local web yielding (omega)                 | 1.500
   Column local web yielding capacity                        |
      [Ra=Rn/omega]                                          | 1017642.667 N
   Interaction ratio in web local yielding                   |
      [Fu/Ra]                                                | 0.939
                                                             |
Column Local Web Crippling Check:                            |
   Nominal strength in column web crippling (Rn)             | 2823518.909 N
   ASD factor in web crippling (omega)                       | 2.000
   Column web crippling capacity                             |
      [Ra=Rn/omega]                                          | 1411759.455 N
```

```
   Interaction ratio in web crippling                                   |
      [Fu/Ra]                                                           | 0.677
                                                                        |
Column Panel Shear Check:                                               |
   Nominal strength of column in panel shear                            | 2645601.487 N
   ASD factor for column panel shear (omega)                            | 1.670
   Column panel shear capacity                                          |
      [Ra=Rn/omega]                                                     | 1584192.507 N
   Interaction ratio for column panel shear                             |
      [Fu/Ra]                                                           | 0.603
                                                                        |
Column Web Shear Buckling Check:                                        |
   Minimum thickness of web to prevent shear buckling (tw‘)             | 7.331 mm
   Interaction ratio for thickness of web                               |
      [tw‘/tw]                                                          | 0.270
```

3.5 Validation problem 5

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_5
------------------------------------------------------------------------+---------------------
Design Summary                                                          |
------------------------------------------------------------------------+---------------------
   Connection is OK                                                     |
   Maximum interaction ratio                                            | 0.996
------------------------------------------------------------------------+---------------------
Design Inputs                                                           |
------------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                       | 4 BOLT - STIFFENED
   Design method                                                        | ASD
   Shear force in connection (V)                                        | 30000.000 N
   Column axial force                                                   | 540000.000 N
   Modulus of elasticity of steel                                       | 200000.000 MPa
   Yield strength of beam (Fyb)                                         | 345.000 MPa
   Tensile strength of beam                                             | 450.000 MPa
   Yield strength of column                                             | 345.000 MPa
   Tensile strength of column                                           | 450.000 MPa
   Yield strength of plate                                              | 250.000 MPa
   Tensile strength of plate                                            | 400.000 MPa
   Tensile strength of weld                                             | 482.000 MPa
   Material overstrength factor for beam (Ry)                           | 1.100
   End plate dimensions (b x d x tp)                                    | 400 x 900 x 35 mm
   Bolt diamter                                                         | 30.000 mm
   Bolt grade                                                           | ASTM A490
   Thickness of beam web to plate fillet weld                           | 10.000 mm
   Thickness of flange weld reinforing fillet weld                      | 8.000 mm
   Thickness of end plate stiffener (tsp)                               | 16.000 mm
   Length of end plate stiffener                                        | 400.000 mm
   Thickness of plate stiffener to beam flange weld                     | 12.000 mm
   Beam section property                                                | W460X60
      Depth (d)                                                         | 455.000 mm
      Flange width                                                      | 153.000 mm
```

```
      Web thickness                                                   | 8.000 mm
      Flange thickness (tf)                                           | 13.300 mm
      Fillet dimension                                                | 23.500 mm
      Plastic section modulus (Zx)                                    | 1280000.000 mm^3
   Column section property                                            | W360X196
      Depth                                                           | 373.000 mm
      Flange width                                                    | 373.000 mm
      Web thickness                                                   | 16.400 mm
      Flange thickness                                                | 26.200 mm
      Fillet dimension                                                | 41.400 mm
   Thickness of column stiffener (tsc)                                | 18.000 mm
   Width of column stiffener                                          | 150.000 mm
   Column stiffener clip dimension                                    | 25.000 mm
   Column stiffener to flange weld                                    | 14.000 mm
   Column stiffener to web weld                                       | 6.000 mm
   Thickness of column doubler plate                                  | 35.000 mm
   Height of column doubler plate                                     | 850.000 mm
   Column doubler to flange weld                                      | 10.000 mm
----------------------------------------------------------------------+--------------------
Design calculations                                                   |
----------------------------------------------------------------------+--------------------
                                                                      |
Connection design forces:                                             |
   Required plastic moment capacity of beam                           |
      [Mpe=1.1*Ry*Fyb*Zx]                                             | 534336000.000 N mm
   Distance from the face of the column to the plastic hinge (Lp)     | 435.000 mm
   Required moment capacity of connection                             |
      [Muc=Mpe+1.5*V*Lp]                                              | 547386000.000 N mm
   Flange force                                                       |
      [Fu=Muc/(d-tf)]                                                 | 1239270.998 N
                                                                      |
Bolt Tension check:                                                   |
   Distance of the centerline of the beam compression flange to the   |
      1st tension bolt row (h0)                                       | 608.350 mm
      2nd tension bolt row (h1)                                       | 391.700 mm
   Bolt nominal tensile stress                                        | 780.000 MPa
   Required bolt diameter                                             |
      [db‘=sqrt(2*Muc/(PI*phi_bv*Fnt*(h0+h1)))]                       | 29.891 mm
   Interaction ratio in bolt tension                                  |
      [db‘/db]                                                        | 0.996
                                                                      |
Bolt Shear Rupture Check:                                             |
   Number of bolts provided at compression flange (nc)                | 4.000
   Nominal shear strength of bolt (Vb)                                | 331559.550 N
   Nominal shear strength of connectionType                           |
      [Rn=nc*Vb]                                                      | 1326238.200 N
   ASD factor in bolt shear (omega)                                   | 2.000
   Bolt shear capacity                                                |
      [Ra=Rn/omega]                                                   | 663119.100 N
   Interaction ratio in bolt shear                                    |
      [Vu/Ra]                                                         | 0.045
                                                                      |
Bolt Bearing on Plate:                                                |
```

```
   Nominal strength in bolt bearing on plate (Rn)                    | 3561600.000 N
   ASD factor in bolt bearing (omega)                                | 2.000
   Bolt bearing on plate capacity                                    |
      [Ra=Rn/omega]                                                  | 1780800.000 N
   Interaction ratio in bolt bearing on plate                        |
      [Vu/Ra]                                                        | 0.017
                                                                     |
Bolt Bearing on Column Flange:                                       |
   Nominal strength in bolt bearing on flange (Rn)                   | 3395520.000 N
   ASD factor in bolt bearing (omega)                                | 2.000
   Bolt bearing on flange capacity                                   |
      [Ra=Rn/omega]                                                  | 1697760.000 N
   Interaction ratio in bolt bearing on flange                       |
      [Vu/Ra]                                                        | 0.018
                                                                     |
Plate Thickness Calculation:                                         |
   Bolt tensile strength (Pt)                                        | 551070.000 N
   No prying design moment                                           |
      [Mnp= 2*Pt*(h0+h1)]                                            | 1102195107.000 N mm
   ASD factor in bolt rupture (omega)                                | 2.000
   ASD factor in bending (omega_b)                                   | 1.670
   Yield line parameter for plate (Yp)                               | 6169.145 mm
   Required minimum plate thickness                                  |
      [tp‘=sqrt(1.11*omega_b*Mnp/(omega*Fyp*Yp))]                    | 25.737 mm
   Interaction ratio for plate thickness                             |
      [tp‘/tp]                                                       | 0.735
                                                                     |
Beam Web to End Plate Weld Tension Check:                            |
   Required tension strength of weld at beam web (fw)                | 2760.000 N/mm
   Nominal strength of weld at beam web (fn)                         | 6133.932 N/mm
   ASD factor in weld (omega)                                        | 2.000
   Beam web weld capacity                                            |
      [fa=fn/omega]                                                  | 3066.966 N/mm
   Interaction ratio for beam web weld in tension                    |
      [fw/fa]                                                        | 0.900
                                                                     |
Beam Web to End Plate Weld Shear Check:                              |
   Total weld length                                                 | 431.600 mm
   Nominal strength of weld at beam web (Rn)                         | 882468.350 N
   ASD factor in weld (omega)                                        | 2.000
   Beam web weld capacity                                            |
      [Ra=Rn/omega]                                                  | 441234.175 N
   Interaction ratio for beam web weld in shear                      |
      [Vu/Ra]                                                        | 0.068
                                                                     |
End Plate Stiffener Thickness Check:                                 |
   Required minimum stiffener thickness (tsp‘)                       | 14.081 mm
   Interaction ratio for stiffener thickness                         |
      [tsp‘/tsp]                                                     | 0.880
                                                                     |
End Plate Stiffener to Flange Weld Check:                            |
   Required strength of stiffener weld                               |
      [Vs=0.6*Fyp*ts]                                                | 2400.000 N/mm
```

```
   Nominal strength of stiffener to flange weld (Rn)                      | 4907.146 N/mm
   ASD factor in weld (omega)                                             | 2.000
   Stiffener to flange weld capacity                                      |
      [Ra=Rn/omega]                                                       | 2453.573 N
   Interaction ratio for stiffener to flange weld                         |
      [Vs/Ra]                                                             | 0.978
                                                                          |
Column Flange Yielding Check:                                             |
   Column yield line parameter (Yc)                                       | 6934.378 mm
   ASD factor in bolt rupture (omega)                                     | 2.000
   ASD factor in flexure (omega_b)                                        | 1.670
   Required minimum column flange thickness                               |
      [tf‘=sqrt(1.11*omega_b*Mnp/(omega*Fyc*Yc))]                         | 20.664 mm
   Interaction ratio for column flange yielding                           |
      [ratio=tf‘/tf]                                                      | 0.789
                                                                          |
Column Web Yielding Check:                                                |
   Bearing length                                                         |
      [N=tf+2*wb]                                                         | 29.300 mm
   Nominal strength of column web in local yielding (Rn)                  | 1066533.000 N
   LRFD factor in local web yielding (omega)                              | 1.500
   Column local web yielding capacity                                     |
      [Ra=Rn/omega]                                                       | 711022.000 N
   Interaction ratio in web local yielding                                |
      [Fu/Ra]                                                             | 0.000
                                                                          |
Column Local Web Crippling Check:                                         |
   Nominal strength in column web crippling (Rn)                          | 2522724.774 N
   ASD factor in web crippling (omega)                                    | 2.000
   Column web crippling capacity                                          |
      [Ra=Rn/omega]                                                       | 1261362.387 N
   Interaction ratio in web crippling                                     |
      [Fu/Ra]                                                             | 0.000
                                                                          |
Column Panel Shear Check:                                                 |
   Nominal strength of column in panel shear                              | 1615715.689 N
   ASD factor for column panel shear (omega)                              | 1.670
   Column panel shear capacity                                            |
      [Ra=Rn/omega]                                                       | 967494.425 N
   Interaction ratio for column panel shear                               |
      [Fu/Ra]                                                             | 0.000
                                                                          |
Column Web Shear Buckling Check:                                          |
   Minimum thickness of web to prevent shear buckling (tw‘)               | 8.470 mm
   Interaction ratio for thickness of web                                 |
      [tw‘/tw]                                                            | 0.516
                                                                          |
Transverse Stiffener Axial Strength:                                      |
   Required strength of column stiffener (Ru)                             | 528248.998 N
   Nominal yield strength of stiffener (Rn)                               | 562500.000 N
   ASD factor in axial strength (omega)                                   | 1.670
   Stiffener axial yield capacity                                         |
      [Ra=Rn/omega]                                                       | 1261362.387 N
```

```
   Interaction ratio for stiffener in axial strength                 |
      [Ru/Ra]                                                        | 0.784
                                                                     |
Transverse stiffener shear strength:                                 |
   Nominal shear strength of stiffener (Rn)                          | 730620.000 N
   ASD factor in stiffener shear strength (omega)                    | 1.670
   Stiffener shear capacity                                          |
      [Ra=Rn/omega]                                                  | 437497.006 N
   Interaction ratio for stiffener in shear                          |
      [Ru/(2*Ra)]                                                    | 0.604
                                                                     |
Transverse Stiffener Thickness Check:                                |
   Minimum required thickness of stiffener (tsc‘)                    | 9.508 mm
   Interaction ratio for stiffener thickness                         |
      [tsc‘/tsc]                                                     | 0.528
                                                                     |
Transverse Stiffener to Flange Weld Check:                           |
   Required strength for stifferner to flange weld (Ps)              | 4050.000 N/mm
   Nominal strength of stiffener to flange weld (Rn)                 | 8587.505 N/mm
   ASD factor weld strength (omega)                                  | 2.000
   Stiffener to flange weld capacity                                 |
      [Ra=Rn/omega]                                                  | 4293.752 N
   Interaction ratio for stiffener to flange weld                    |
      [Ps/Ra]                                                        | 0.943
                                                                     |
Transverse Stiffener to Web Weld Check:                              |
   Nominal strength of stiffener to web weld (Rn)                    | 663936.800 N/mm
   ASD factor weld strength (omega)                                  | 2.000
   Stiffener to web weld capacity                                    |
      [Ra=Rn/omega]                                                  | 331968.400 N/mm
   Interaction ratio for stiffener to web weld                       |
      [Ru/(2*Ra)]                                                    | 0.796
                                                                     |
Doubler Plate Shear Check:                                           |
   Required shear strength of doubler plate (Vd)                     | 271776.574 N
   Nominal shear strength of doubler plate (Rn)                      | 1958250.000 N
   ASD factor for shear (omega)                                      | 1.500
   Doubler plate shear capacity                                      |
      [Ra=Rn/omega]                                                  | 1305500.000 N
   Interaction ratio for doubler plate shear                         |
      [Vd/Ra]                                                        | 0.208
                                                                     |
Doubler Plate Height Check:                                          |
   Minimum required height of doubler plate (wd‘)                    | 773.400 mm
   Interaction ratio for doubler plate height                        |
      [wd‘/wd]                                                       | 0.910
                                                                     |
Doubler Plate Thickness Check:                                       |
   Minimum required thickness of doubler plate (td‘)                 | 8.270 mm
   Interaction ratio for doubler plate thickness                     |
      [td‘/td]                                                       | 0.236
                                                                     |
Doubler Plate to Flange Weld Check:                                  |
```

```
   Nominal strength of doubler to flange weld (Rn)                      | 1737947.400 N
   ASD factor weld strength (omega)                                     | 2.000
   Doubler to flange weld capacity                                      |
      [Ra=Rn/omega]                                                     | 868973.700 N
   Interaction ratio for plate to weld thickness                        |
      [Vd/Ra]                                                           | 0.313
```

3.6 Validation problem 6

```
Osoconn v1.1
Connection code : MC001AM10
Connection ID : MC001_6
-----------------------------------------------------------------------+---------------------
Design Summary                                                         |
-----------------------------------------------------------------------+---------------------
   Connection is OK                                                    |
   Maximum interaction ratio                                           | 0.978
-----------------------------------------------------------------------+---------------------
Design Inputs                                                          |
-----------------------------------------------------------------------+---------------------
   Extended plate moment connection configuration                      | 8 BOLT - STIFFENED
   Design method                                                       | LRFD
   Shear force in connection (V)                                       | 510000.000 N
   Column axial force                                                  | 8050000.000 N
   Modulus of elasticity of steel                                      | 200000.000 MPa
   Yield strength of beam (Fyb)                                        | 345.000 MPa
   Tensile strength of beam                                            | 450.000 MPa
   Yield strength of column                                            | 345.000 MPa
   Tensile strength of column                                          | 450.000 MPa
   Yield strength of plate                                             | 250.000 MPa
   Tensile strength of plate                                           | 400.000 MPa
   Tensile strength of weld                                            | 482.000 MPa
   Material overstrength factor for beam (Ry)                          | 1.100
   End plate dimensions (b x d x tp)                                   | 400 x 700 x 35 mm
   Bolt diamter                                                        | 24.000 mm
   Bolt grade                                                          | ASTM A490
   Thickness of beam web to plate fillet weld                          | 12.000 mm
   Thickness of flange weld reinforing fillet weld                     | 8.000 mm
   Thickness of end plate stiffener (tsp)                              | 16.000 mm
   Length of end plate stiffener                                       | 300.000 mm
   Thickness of plate stiffener to beam flange weld                    | 8.000 mm
   Beam section property                                               | W410X67
      Depth (d)                                                        | 409.000 mm
      Flange width                                                     | 179.000 mm
      Web thickness                                                    | 8.760 mm
      Flange thickness (tf)                                            | 14.400 mm
      Fillet dimension                                                 | 24.600 mm
      Plastic section modulus (Zx)                                     | 1350000.000 mm^3
   Column section property                                             | W360X216
      Depth                                                            | 376.000 mm
      Flange width                                                     | 394.000 mm
      Web thickness                                                    | 17.300 mm
      Flange thickness                                                 | 27.700 mm
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      Fillet dimension                                                     | 42.900 mm
   Thickness of column doubler plate                                       | 35.000 mm
   Height of column doubler plate                                          | 850.000 mm
   Column doubler to flange weld                                           | 10.000 mm
-----------------------------------------------------------------------+---------------------
Design calculations                                                        |
-----------------------------------------------------------------------+---------------------
                                                                           |
Connection design forces:                                                  |
   Required plastic moment capacity of beam                                |
      [Mpe=1.1*Ry*Fyb*Zx]                                                  | 563557500.000 N mm
   Distance from the face of the column to the plastic hinge (Lp)          | 335.000 mm
   Required moment capacity of connection                                  |
      [Muc=Mpe+V*Lp]                                                       | 734407500.000 N mm
   Flange force                                                            |
      [Fu=Muc/(d-tf)]                                                      | 1861144.197 N
                                                                           |
Bolt Tension check:                                                        |
   Distance of the centerline of the beam compression flange to the        |
      1st tension bolt row (h1)                                            | 506.800 mm
      2nd tension bolt row (h2)                                            | 441.800 mm
      3rd tension bolt row (h3)                                            | 354.600 mm
      4th tension bolt row (h4)                                            | 289.600 mm
   Bolt nominal tensile stress                                             | 780.000 MPa
   Required bolt diameter                                                  |
      [db`=sqrt(2*Muc/(PI*phi_bv*Fnt*(h1+h2+h3+h4)))]                      | 22.400 mm
   Interaction ratio in bolt tension                                       |
      [db`/db]                                                             | 0.933
                                                                           |
Bolt Shear Rupture Check:                                                  |
   Number of bolts provided at compression flange (nc)                     | 8.000
   Nominal shear strength of bolt (Vb)                                     | 212198.112 N
   Nominal shear strength of connectionType                                |
      [Rn=nc*Vb]                                                           | 1697584.896 N
   LRFD factor in bolt shear (phi)                                         | 0.750
   Bolt shear capacity                                                     |
      [Ra=Rn*phi]                                                          | 1273188.672 N
   Interaction ratio in bolt shear                                         |
      [Vu/Ra]                                                              | 0.401
                                                                           |
Bolt Bearing on Plate:                                                     |
   Nominal strength in bolt bearing on plate (Rn)                          | 4737600.000 N
   LRFD factor in bolt bearing (phi)                                       | 0.750
   Bolt bearing on plate capacity                                          |
      [Ra=Rn*phi]                                                          | 3553200.000 N
   Interaction ratio in bolt bearing on plate                              |
      [Vu/Ra]                                                              | 0.144
                                                                           |
Bolt Bearing on Column Flange:                                             |
   Nominal strength in bolt bearing on flange (Rn)                         | 4547232.000 N
   LRFD factor in bolt bearing (phi)                                       | 0.750
   Bolt bearing on flange capacity                                         |
      [Ra=Rn*phi]                                                          | 3410424.000 N
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   Interaction ratio in bolt bearing on flange                  |
      [Vu/Ra]                                                   | 0.150
                                                                |
Plate Thickness Calculation:                                    |
   Bolt tensile strength (Pt)                                   | 352684.800 N
   No prying design moment                                      |
      [Mnp= 2*Pt*(h1+h2+h3+h4)]                                 | 1123512698.880 N mm
   LRFD factor in bolt rupture (phi)                            | 0.750
   LRFD factor in bending (phi_b)                               | 0.900
   Yield line parameter for plate (Yp)                          | 7870.383 mm
   Required minimum plate thickness                             |
      [tp‘=sqrt(1.11*phi*Mnp/(phi_b*Fyp*Yp))]                   | 22.982 mm
   Interaction ratio for plate thickness                        |
      [tp‘/tp]                                                  | 0.657
                                                                |
Beam Web to End Plate Weld Tension Check:                       |
   Required tension strength of weld at beam web (fw)           | 3022.200 N/mm
   Nominal strength of weld at beam web (fn)                    | 7360.718 N/mm
   LRFD factor for weld (phi)                                   | 0.750
   Beam web weld capacity                                       |
      [fa=fn*phi]                                               | 5520.539 N/mm
   Interaction ratio for beam web weld in tension               |
      [fw/fa]                                                   | 0.547
                                                                |
Beam Web to End Plate Weld Shear Check:                         |
   Total weld length                                            | 355.200 mm
   Nominal strength of weld at beam web (Rn)                    | 871509.059 N
   LRFD factor for weld (phi)                                   | 0.750
   Beam web weld capacity                                       |
      [Ra=Rn*phi]                                               | 653631.794 N
   Interaction ratio for beam web weld in shear                 |
      [Vu/Ra]                                                   | 0.780
                                                                |
End Plate Stiffener Thickness Check:                            |
   Required minimum stiffener thickness (tsp‘)                  | 12.089 mm
   Interaction ratio for stiffener thickness                    |
      [tsp‘/tsp]                                                | 0.756
                                                                |
End Plate Stiffener to Flange Weld Check:                       |
   Required strength of stiffener weld                          |
      [Vs=0.6*Fyp*ts]                                           | 2400.000 N/mm
   Nominal strength of stiffener to flange weld (Rn)            | 3271.430 N/mm
   LRFD factor for weld (phi)                                   | 0.750
   Stiffener to flange weld capacity                            |
      [Ra=Rn*phi]                                               | 2453.573 N
   Interaction ratio for stiffener to flange weld               |
      [Vs/Ra]                                                   | 0.978
                                                                |
Column Flange Yielding Check:                                   |
   Column yield line parameter (Yc)                             | 4312.191 mm
   LRFD factor in bolt rupture (phi)                            | 0.750
   LRFD factor in flexure (phi_b)                               | 0.900
   Required minimum column flange thickness                     |
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      [tf'=sqrt(1.11*phi*Mnp/(phi_b*Fyc*Yc))]                        | 26.430 mm
   Interaction ratio for column flange yielding                      |
      [ratio=tf'/tf]                                                 | 0.954
                                                                     |
Column Web Yielding Check:                                           |
   Bearing length                                                    |
      [N=tf+2*wb]                                                    | 30.400 mm
   Nominal strength of column web in local yielding (Rn)             | 2135529.300 N
   LRFD factor in local web yielding (phi)                           | 1.000
   Column local web yielding capacity                                |
      [Ra=Rn*phi]                                                    | 2135529.300 N
   Interaction ratio in web local yielding                           |
      [Fu/Ra]                                                        | 0.872
                                                                     |
Column Local Web Crippling Check:                                    |
   Nominal strength in column web crippling (Rn)                     | 2817941.124 N
   LRFD factor in web crippling (phi)                                | 0.750
   Column web crippling capacity                                     |
      [Ra=Rn*phi]                                                    | 2113455.843 N
   Interaction ratio in web crippling                                |
      [Fu/Ra]                                                        | 0.881
                                                                     |
Column Panel Shear Check:                                            |
   Nominal strength of column in panel shear                         | 1592127.677 N
   LRFD factor for column panel shear (phi)                          | 0.900
   Column panel shear capacity                                       |
      [Ra=Rn*phi]                                                    | 1432914.910 N
   Interaction ratio for column panel shear                          |
      [Fu/Ra]                                                        | 0.000
                                                                     |
Column Web Shear Buckling Check:                                     |
   Minimum thickness of web to prevent shear buckling (tw')          | 7.947 mm
   Interaction ratio for thickness of web                            |
      [tw'/tw]                                                       | 0.459
                                                                     |
Doubler Plate Shear Check:                                           |
   Required shear strength of doubler plate (Vd)                     | 428229.287 N
   Nomimal shear strength of doubler plate (Rn)                      | 1974000.000 N
   LRFD factor for shear (phi)                                       | 1.000
   Doubler plate shear capacity                                      |
      [Ra=Rn*phi]                                                    | 1974000.000 N
   Interaction ratio for doubler plate shear                         |
      [Vd/Ra]                                                        | 0.217
                                                                     |
Doubler Plate Height Check:                                          |
   Minimum required height of doubler plate (wd')                    | 736.400 mm
   Interaction ratio for doubler plate height                        |
      [wd'/wd]                                                       | 0.866
                                                                     |
Doubler Plate Thickness Check:                                       |
   Minimum required thickness of doubler plate (td')                 | 7.947 mm
   Interaction ratio for doubler plate thickness                     |
      [td'/td]                                                       | 0.227
```

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                                                                       |
Doubler Plate to Flange Weld Check:                                    |
   Nominal strength of doubler to flange weld (Rn)                     | 1737947.400 N
   LRFD factor for weld strength (phi)                                 | 0.750
   Doubler to flange weld capacity                                     |
      [Ra=Rn*phi]                                                      | 1303460.550 N
   Interaction ratio for plate to weld thickness                       |
      [Vd/Ra]                                                          | 0.329
```