Benchmark study of a bolted flange plate moment connection

Prepared for IDEA StatiCa.

by

Nicolás Leiva, Research Assistant
Ricardo Herrera Mardones, Ph. D., Associate Professor
Department of Civil Engineering, Universidad de Chile

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1.1. Bolted Flange Plated - Fully Restrained Moment Connection

1.1.1. Description

This study presents the results of the verification of a Bolted Flange Plate (BFP) fully restrained moment connection, shown in Fig.1. It compares the results of hand calculations according to AISC 360-16 with the results of CBFEM in IDEA StatiCa Connection. Beam flanges and web are bolted to plates connected by welds to the column flange. The beam is subjected to a concentrated load at a variable distance from the column centerline, which depends on the failure mode that controls the design of the connection.

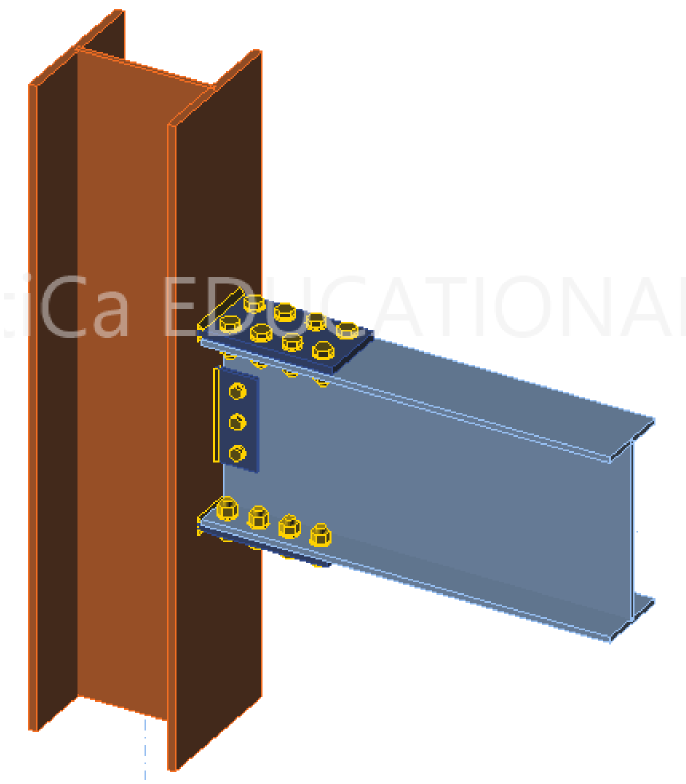

Fig.1. Bolted Flange Plated – Fully restrained moment connection.

1.1.2. Analytical model

The flange components are analyzed to resist bending stresses through:

- strength of bolts in tension and shear,
- strength of flanges and flange plates in bearing,
- tearout and block shear,
- fillet weld,
- compression in flange plate,
- flange local bending of columns,
- web local yielding of columns,
- web local crippling.

Web components are designed to resist shear:

- strength of bolts,
- shear strength of plates,
- welds,
- column flange rupture.

All the verifications are executed with AISC provisions to resist the maximum possible moment, which is achieved by modifying the position of the load. Details related to these verifications and all the dimensions for the connection components are presented in section 1.1.6.

Strength of bolts in tension and shear

The spacing between bolts and the edge distance depends on the nominal diameter of bolts. Basic resistance of bolts depends on the specification of the bolt material and geometry, according to Table J3.2 in AISC.

Strength of flanges and flange plates

The plates are verified to resist bearing and tearout, which depends on the nominal diameter of bolts, thickness of plate, spacing of bolts, and quality of steel according to J3.10. They are also verified against block shear, analyzing 3 possible case scenarios. The components resist the force in shear and tension utilizing combinations of the gross and net area in tension and shear, according to J4.3. The minimum resistance characterizes the critical component and determines the resistance of the plate.

Fillet weld

Based on the thickness of connected plates, minimum and maximum weld leg sizes are established. Effective thickness and effective length are calculated according to J2.2, which gives the resistance of the component.

Compression in flange plate

The flange plates are verified against buckling produced by compression forces induced by the beam bending moment.

Column flange local bending

The column flanges are verified against local bending induced by the concentrated tensile load produced by the beam bending moment, which depends on the flange thickness and yield strength of steel, conforming to J10.1.

Column web local yielding

The column is verified to resist the concentrated compression force, which is a consequence of bending moment acting in the beam. It depends on the flange thickness, the column thickness, and the distance from the outer face of the flange to the toe of the fillet weld. Case

a) is the corresponding case for the J.10.2 mode of failure, because the force is not applied near the end of the column.

Column web local crippling

The bending force in the beam generates concentrated loads in columns that are verified according to case a) in J.10.3.

Column flange rupture

The column flange is verified against rupture, produced by transmission of shear stress through the plate connecting the beam web to the column flange.

1.1.3. Numerical design model

The plates are modelled by 4-node shell elements with 6 degrees of freedom per node. Deformations of the element include membrane and flexural contributions. A nonlinear elastic plastic material constitutive relationship is assumed and stresses in each layer are monitored at every integration point. Bolts act in shear only. The load is applied as a concentrated load that generates bending moment in the connection as a consequence of the distance from the point of application to the column axis. As can be observed in Fig.1, the plates are connected to the beam using 4-rows of two bolts per flange and a single row of three bolts for the web. The distribution of bolts in the flanges is shown in Fig. 2. The bolt spacing is 3 [in] and the distances to the edge of the plate and the face of the column are 1.5 [in] and 2 [in], respectively. Transverse bolt gage is 4 [in]. For the web, the bolt spacing is 3 [in] and the edge distance is 1.5 [in], as shown in Fig. 3. The bolt diameter is 7/8 [in] for the benchmark example. Flange and web plates are welded to the column using continuous fillet welds on both sides of the plate. Two 1/4 [in] fillets are used for the web plate and two 3/8 [in] fillets for each flange plate. The flange plates are 7 [in] wide and 12.5 [in] long. The web plate is 5 [in] wide and 9 [in] long.

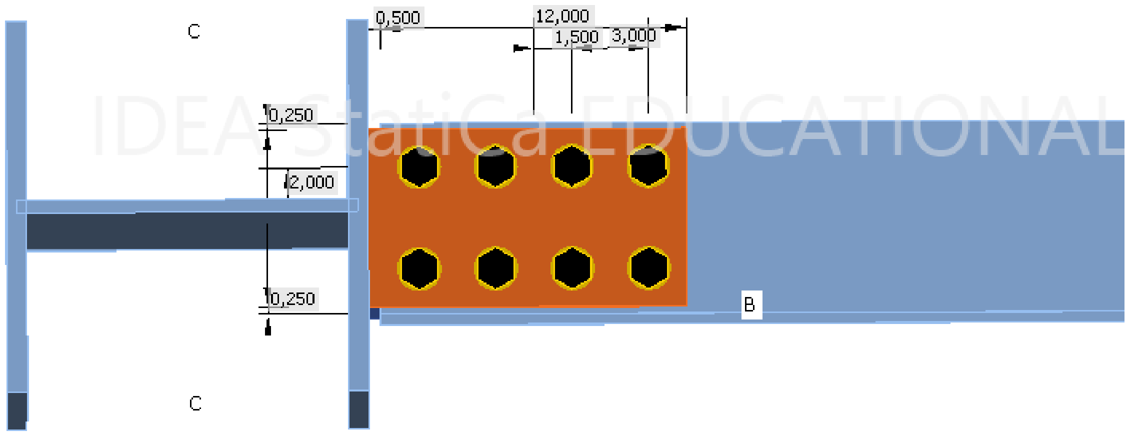

Fig. 2. Flange plate bolt distribution.

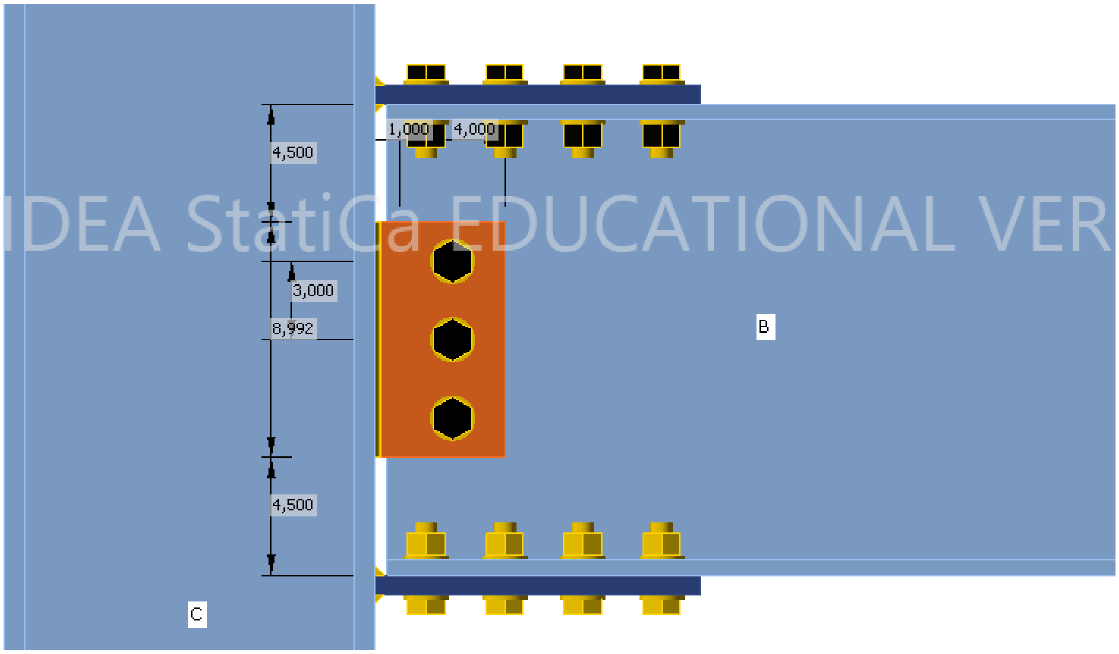

Fig. 3. Web plate bolt distribution.

1.1.4. Global behavior

Comparison of global behavior of the joint for the benchmark example is done through the moment-rotation response of the connection. AISC calculations give the maximum resistance and failure mechanism, but no deformation or stiffness for the connection can be calculated. Since the connection is classified as fully rigid per AISC, a rigid plastic response is assumed for the graph. Fig. 4 shows a comparison of both methods, where it is possible to appreciate that the CBFEM method predicts a smaller resistance than the AISC equations for this case.

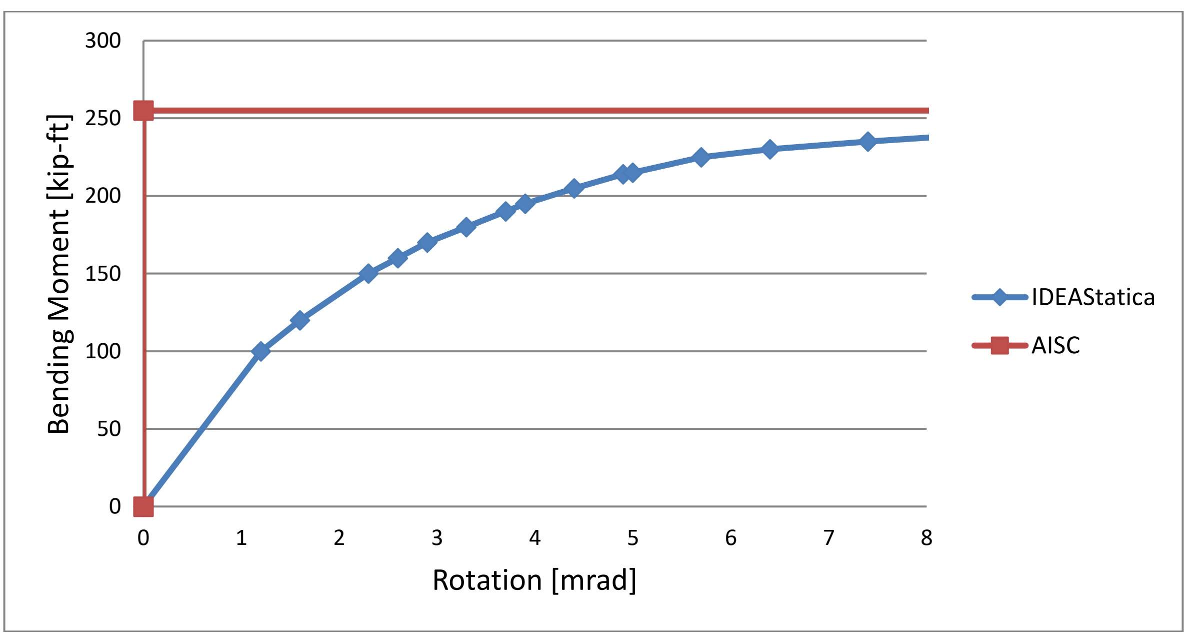

Fig. 4. Moment-rotation diagram.

1.1.5. Verification of resistance

The design resistances obtained using the AISC method and CBFEM were compared. The benchmark example (case 8, in Table 1) calculations according to AISC are summarized in 1.1.6, and the variations from this base configuration are described in Table 1. The comparison is focused on the resistance and the critical component. The flange plates are 7 [in] wide and 12.5 [in] long. The web plate is 5 [in] wide and 9 [in] long. The variations from the benchmark example include: flange plate thickness, web plate thickness, bolt diameter, flange plate weld size, web plate weld, and column size.

Table 1. Comparison of design resistance and failure mode between AISC and CBFEM.

		AISC		CBFEM		
Case	Case Description	Failure Mode	ϕMn [kip-ft]	Failure Mode	ϕMn [kip-ft]	CBFEM/ AISC
1	Column W12 × 65	Column flange local bending	161	Weld in compression	178	1.11
2	Column W12 × 65 and 7/16'' flange plate weld	Column flange local bending	161	Weld in compression	193.7	1.20
3	Column W12 × 65 and 1/2'' flange plate weld	Column flange local bending	161	Panel zone yielding	203.5	1.26
4	1 Fillet weld in web	Flange plate fracture	255	Weld in compression	214.3	0.84
5	Distance to edge in web plate 1,2''	Flange plate fracture	255	Weld in compression	215	0.84
6	Web plate 1/4'' thick	Web plate fracture	238*	Weld in compression	212	0.89
7	5/8'' diameter bolts in web	Web plate bolts fracture	226*	Weld in compression	210.9	0.93
8	Benchmark example	Flange plate fracture	255	Weld in compression	214.6	0.84
9	7/16'' flange plate weld	Flange plate fracture	255	Flange plate fracture	248.8	0.98
10	1/2'' flange plate weld	Flange plate fracture	255	Flange plate fracture	249.6	0.98
11	Column W12 x 96	Flange plate fracture	255	Weld in compression	226.5	0.89
12	Column W12 x 96 and 7/16'' flange plate weld	Flange plate fracture	255	Flange plate fracture	250	0.98
13	7/16'' flange plate weld and 10 - 3/4'' diameter bolts	Flange plate yielding	266	Flange plate fracture	251.4	0.95
14	10 - 3/4'' diameter bolts per flange	Flange plate yielding	266	Weld in compression	215.8	0.81
15	Flange plate 7/8'' thick	Column flange local bending	269	Weld in compression	212.3	0.79

*: calculated as the shear strength multiplied by the ratio between the bending strength and the shear force for the benchmark case

The results plotted in Figs. 5 and 6 show that CBFEM model gives lower resistance than AISC in all cases, except when column flange bending is the controlling failure mode according to AISC calculations. The green dots in Fig. 5 are the ratio between the CBFM and the AISC resistance estimations. The differences between both methods stay within 10%, except for cases when the failure of the weld in compression is the controlling failure mode according to CBFEM model calculations or when column flange bending is the controlling failure mode according to AISC calculations.

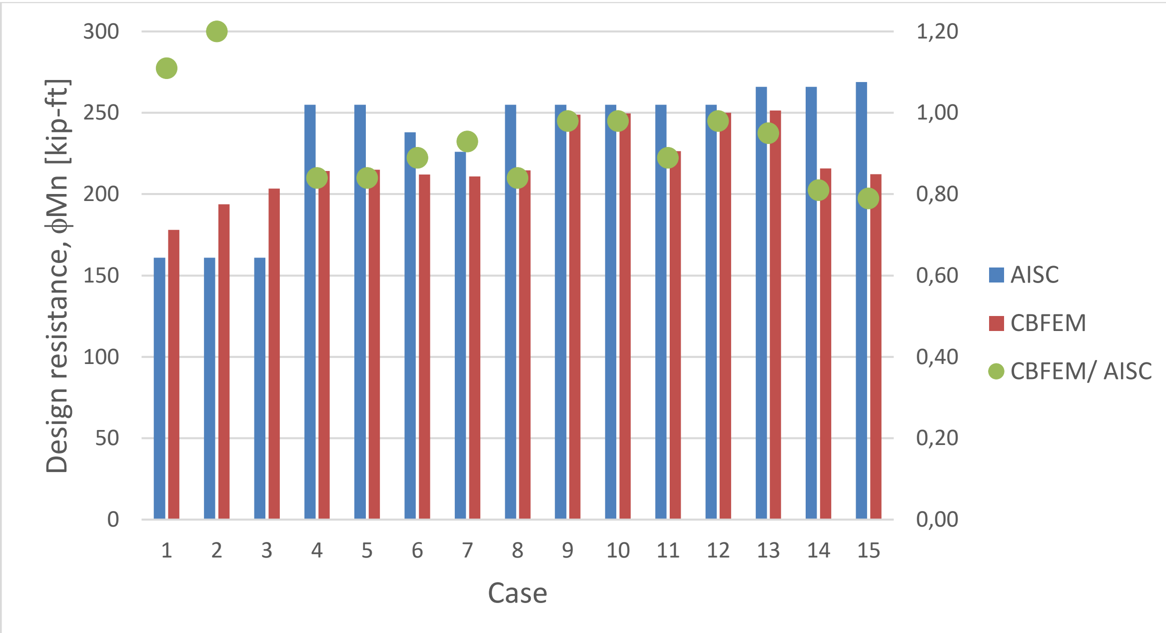

Fig. 5. Design resistance comparison

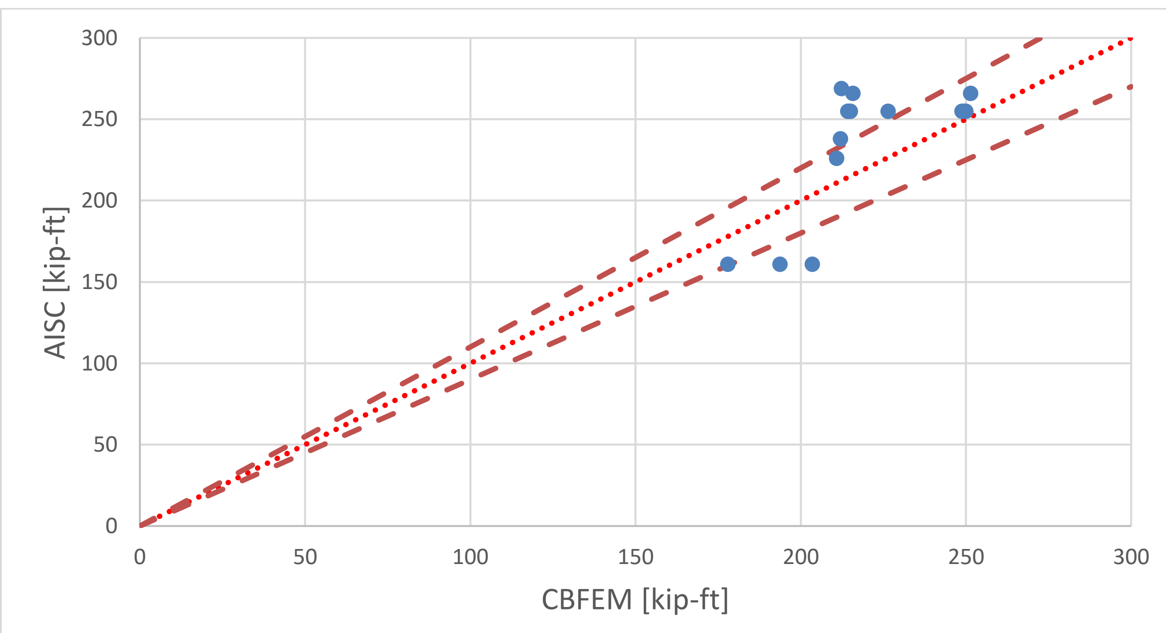

Fig. 6. Comparison of CBFEM and AISC.

Based on the previous observation, the analyses are repeated considering butt welds between the flange plate and the column flange, to eliminate the failure mode associated to weld fracture in this component. The results are presented in Table 2, showing better agreement between both methods in the resistance value and the failure mode. The case numbers correspond to the case number of the previous parametric analysis. Cases not present are those that considered variations on the fillet weld size.

Table 2. Comparison of design resistance and failure mode considering butt welds between the beam flange plate and the column.

		AISC		CBFEM		
Case	Case Description	Failure Mode	ϕMn [kip-ft]	Failure Mode	ϕMn [kip-ft]	CBFEM/ AISC
1	Column W12 × 65	Column flange local bending	161	Flange plate yielding	201.5	1.25
5	Distance to edge in web plate 1,2"	Flange plate fracture	255	Flange plate fracture	252.1	0.99
6	Web plate 1/4" thick	Web plate fracture	238*	Flange plate fracture	248.8	1.05
7	5/8" diameter bolts in web	Web plate bolts fracture	226*	Flange plate fracture	247.9	1.10
8	Benchmark example with butt welds	Flange plate fracture	255	Flange plate fracture	251.3	0.99
11	Column W12 x 96	Flange plate fracture	255	Flange plate fracture	254.1	1.00
14	10 - 3/4" diameter bolts per flange	Flange plate compression yielding	266	Flange plate fracture	252.9	0.95
15	Flange plate 7/8" thick	Column flange local bending	269	Flange plate fracture	285.4	1.06

*: calculated as the shear strength multiplied by the ratio between the bending strength and the shear force for the benchmark case

Figs. 7 and 8 illustrate the improvement in the agreement. This seems to indicate that the criteria used by the program to evaluate the fillet weld resistance is too conservative and could be revised. The difference in strength between AISC and CBFEM is due to the way the demand is determined in both methods. In the case of AISC, an average demand on the fillet welds on either side of the flange plate is obtained by dividing the bending moment by the moment arm between the flange plates. In CBFEM, the demand is determined, from the FEM results, for each fillet weld separately, and it is affected by the bending of the column flange. If continuity plates (i.e. stiffeners in the column aligned with the flange plates) are introduced in the benchmark case with fillet welds, the resistance increases to 245 [kip-ft], or 96% of the resistance calculated with AISC. An issue that is not resolved is the difference when the column flange local bending is the controlling failure mode according to AISC calculations. The model does not seem to capture this failure mode properly. However, if continuity plates are used, as is often the case in seismic design, this mode of failure is prevented and a better estimation of the connection capacity is achieved.

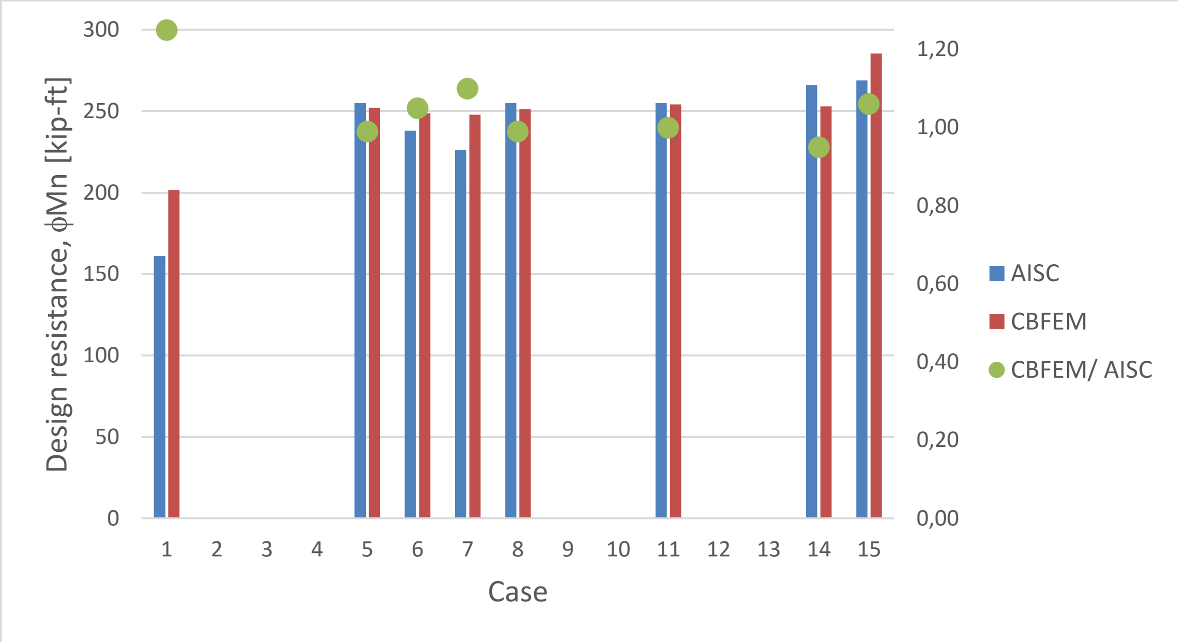

Fig. 7. Design resistance comparison for butt weld configuration.

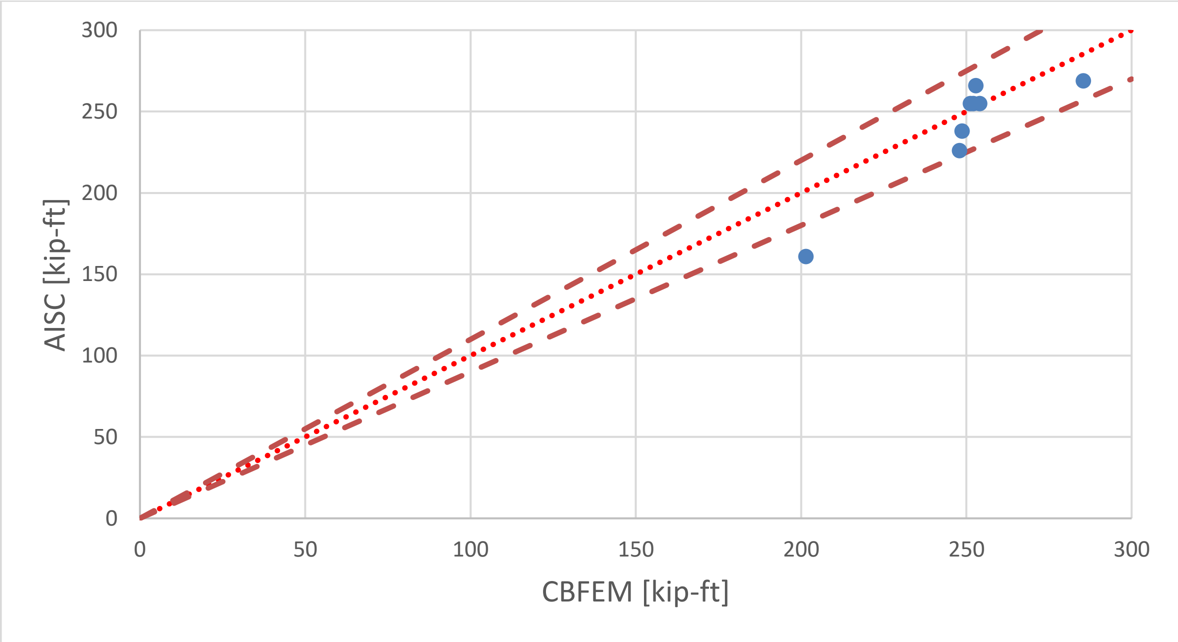

Fig. 8.Verification of CBFEM to AISC for butt weld configuration.

1.1.6. Benchmark example

Inputs

- Beam W18×50, ASTM A992 steel
- Column W14×99, ASTM A992 steel
- Flange plate - PL 3/4''×7''×1'-1/2'', ASTM A36 steel
- Web plate – PL 3/8''×5''×9'', ASTM A36 steel
- Flange plate weld size 3/8'', 70 ksi electrode
- Web plate weld size 1/4'', 70 ksi electrode
- Bolt diameter 7/8'', type A325-N (threads included in the shear transfer plane), standard holes.

Outputs

- Design resistance in bending ϕM_n = 214.6 [kip-ft]
- Vertical shear force V =42 [kip]
- Failure mode: Weld in compression (Fig. 9)
- Flange plate equivalent strain: 0.0062 < 0.05
- Utilization of bolts: 70.2%
- Utilization of welds in tension: 96.8%

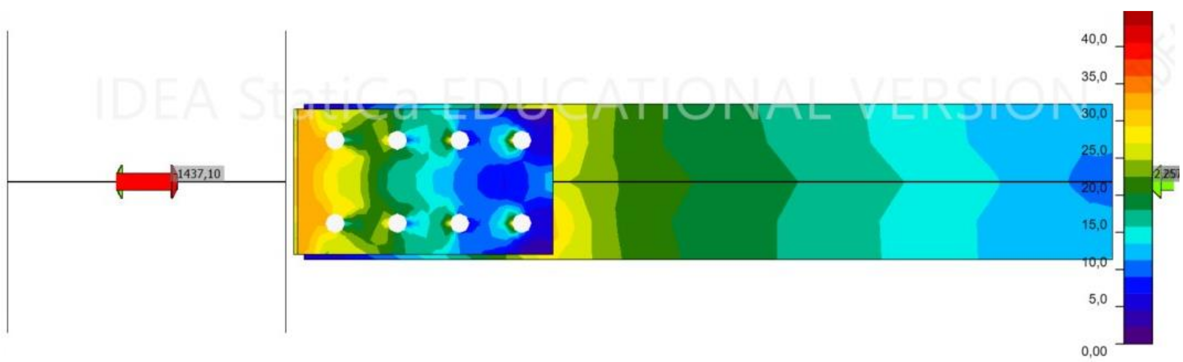

Fig. 9. Failure mode for benchmark example: Weld in compression.

Calculations according to AISC

1. Beam flange bolt shear strength ($\phi = 0.75$):

$d_b = \frac{7}{8}$", $F_{nv} = 54$ ksi, $\phi = 0.75$

$A_b = 0.6\ in^2$, $R_{nv} = F_{nv} A_b = 32.5$ kip, $\boldsymbol{\phi R_{nv} = 24}$ **kip**

$d = 18$ in

$\boldsymbol{\phi M_n = 8\phi R_{nv}\ d = 292}$ **kip-ft**

2. Flange plate tensile strength ($\phi = 0.75$):

$A_e = t_{fp}\ (7\ in - 2(d_h + {}^1/_{16}"))= 3.8\ in^2$

$R_n = F_{up}\ A_e = 217.5$ kip

$\boldsymbol{\phi R_n = 163.1}$ **kip**

$\boldsymbol{\phi M_n = \phi R_n\ (d + t_{fp}) = 255}$ **kip-ft**

3. Flange plate fillet weld fracture ($\phi = 0.75$):

$w = \frac{3}{8}$" $l_w = 7$ in

$t_e = 0.707\ w = 0.265$ in

$F_{nw} = 0.6\ F_{E70}\ (1 + 0.5\ \sin^{1.5}\theta) = 63$ ksi

$R_{nw} = F_{nw}\ t_e\ 2l_w = 233.8$ kip

$\boldsymbol{\phi R_{nw} = 175.4}$ **kip**

$\boldsymbol{\phi M_n = \phi R_{nw}\ (d + t_{fp}) = 274}$ **kip-ft**

4. Flange plate bearing strength ($\phi = 0.75$):

$t_{fp} = \frac{3}{4}$" $F_{yp} = 36$ ksi $F_{up} = 58$ ksi

$R_{nbfp} = 2.4\ d_b\ t_{fp}\ F_{up} = 91.4$ kip $\boldsymbol{\phi R_{nbfp} = 68.5}$ **kip**

$\boldsymbol{\phi M_n = 8\phi R_{nbfp}\ (d + t_{fp}) = 856}$ **kip-ft**

5. Flange plate tearout strength ($\phi = 0.75$):

Interior bolt

$s = 3$ in $d_h = d_b + {}^1/_{16}" = 0.94$ in $l_c = s - d_h = 2.1$ in

$R_{nto\_i} = 1.2\ l_c\ t_{fp}\ F_{up} = 107.7$ kip $\boldsymbol{\phi R_{nto\_i} = 80.7}$ **kip**

Exterior bolt

$l_c = 1.5 \text{ in} - d_h/2 = 1 \text{ in}$

$R_{nto\_e} = 1.2\, l_c\, t_{fp}\, F_{up} = 53.8$ kip   **$\phi R_{nto\_e} = 40.4$ kip**

$\phi M_n = (6\phi R_{nto\_i} + 2\phi R_{nto\_e})(d + t_{fp}) = 883$ kip-ft

6. Beam flange bearing strength ($\phi = 0.75$):

$t_{fb} = 0.57$ in $F_{yb} = 50$ ksi $F_{ub} = 65$ ksi

$R_{nbfb} = 2.4\, d_b\, t_{fb}\, F_{ub} = 77.8$ kip   **$\phi R_{nbfb} = 58.4$ kip**

$\phi M_n = 8\phi R_{nbfb}\,(d - t_{fb}) = 678$ kip-ft

7. Beam flange tearout strength ($\phi = 0.75$):

Interior bolt

$s = 3$ in $d_h = d_b + {}^1/_{16}" = 0.94$ in $l_c = s - d_h = 2.1$ in

$R_{nto\_i} = 1.2\, l_c\, t_{fb}\, F_{ub} = 91.7$ kip   **$\phi R_{nto\_i} = 68.8$ kip**

Exterior bolt

$l_c = 1.5 \text{ in} - d_h/2 = 1 \text{ in}$

$R_{nto\_e} = 1.2\, l_c\, t_{fb}\, F_{ub} = 45.8$ kip   **$\phi R_{nto\_e} = 34.4$ kip**

$\phi M_n = (6\phi R_{nto\_i} + 2\phi R_{nto\_e})(d - t_{fb}) = 699$ kip-ft

8. Flange plate block shear strength ($\phi = 0.75$):

Case 1:

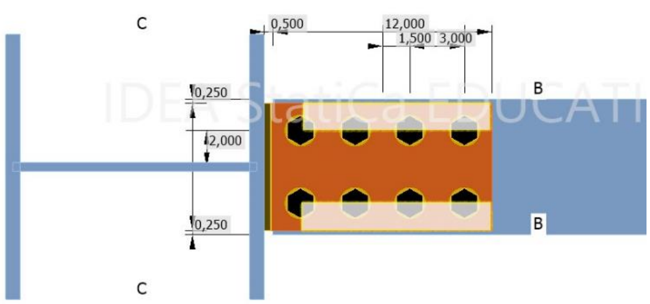

Fig. 10. Case 1 for block shear failure of flange plate.

$l_{gv1} = 3s + 1.5 \text{ in} = 10.5$ in $l_{nv1} = l_{gv1} - 3.5(d_h + {}^1/_{16}") = 7$ in

$A_{gv1} = 2 l_{gv1}\, t_{fp} = 15.8 \text{ in}^2$ $A_{nv1} = 2 l_{nv1}\, t_{fp} = 10.5 \text{ in}^2$

$l_{gt1} = 2{\cdot}1.5 \text{ in} = 3 \text{ in}$

$l_{nt1} = l_{gt1} - (d_h + {}^{1}/_{16}") = 2 \text{ in}$

$A_{gt1} = l_{gt1}\, t_{fp} = 2.3 \text{ in}^2$

$A_{nt1} = l_{nt1}\, t_{fp} = 1.5 \text{ in}^2$

$U_{bs} = 1$

$R_{nbs1} = \min(0.6\, F_{up}\, A_{nv1} + F_{up}\, A_{nt1};\ 0.6\, F_{yp}\, A_{gv1} + F_{up}\, A_{nt1}) = 427.2 \text{ kip}$

$\boldsymbol{\phi R_{nbs1} = 320.4}$ **kip**

Case 2:

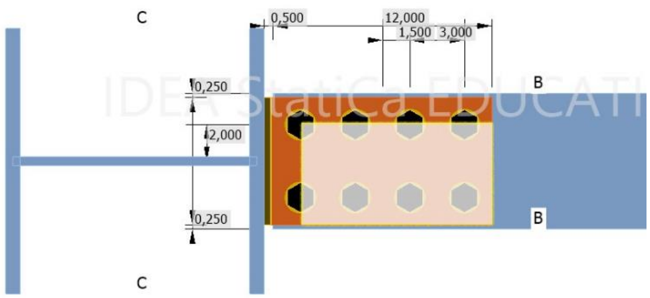

Fig. 11. Case 2 for block shear failure of flange plate.

$A_{gv2} = l_{gv1}\, t_{fp} = 7.9 \text{ in}^2$

$A_{nv2} = l_{nv1}\, t_{fp} = 5.3 \text{ in}^2$

$l_{gt2} = (7 - 1.5) \text{ in} = 5.5 \text{ in}$

$l_{nt2} = l_{gt2} - 1.5(d_h + {}^{1}/_{16}") = 4 \text{ in}$

$A_{gt2} = l_{gt1}\, t_{fp} = 4.1 \text{ in}^2$

$A_{nt2} = l_{nt2}\, t_{fp} = 3 \text{ in}^2$

$U_{bs} = 1$

$R_{nbs2} = \min(0.6\, F_{up}\, A_{nv2} + F_{up}\, A_{nt2};\ 0.6\, F_{yp}\, A_{gv2} + F_{up}\, A_{nt2}) = 344.1 \text{ kip}$

$\boldsymbol{\phi R_{nbs2} = 258.1}$ **kip**

$\boldsymbol{\phi M_n = \min(\phi R_{nbs1}, \phi R_{nbs2})(d + t_{fp}) = 403}$ **kip-ft**

9. Beam flange block shear strength ($\phi = 0.75$):

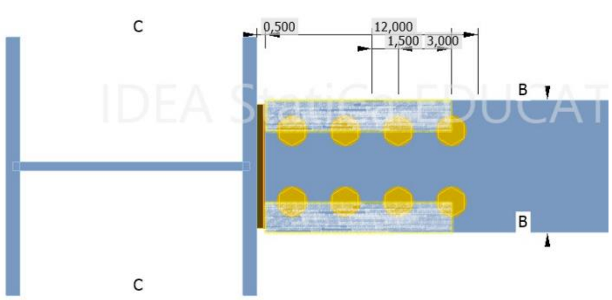

Fig. 12. Case for block shear failure of beam flange.

l_{gv} = 3s + 1.5 in = 10.5 in $\quad$ $l_{nv} = l_{gv}$ -3.5(d_h + $^1/_{16}$") = 7 in

$A_{gv} = 2l_{gv}\ t_{fb}$ = 12 in^2 $\quad$ $A_{nv} = 2l_{nv}\ t_{fb}$ = 8 in^2

l_{gt} = (7.5 - 4) in = 2.5 in $\quad$ $l_{nt} = l_{gt}$ - (d_h + $^1/_{16}$") = 2.5 in

$A_{gt} = l_{gt}\ t_{fb}$ = 2 in^2 $\quad$ $A_{nt} = l_{nt}\ t_{fb}$ = 1.4 in^2

U_{bs} = 1

R_{nbs} = min (0.6 F_{ub} A_{nv} + F_{ub} A_{nt}; 0.6 F_{yb} A_{gv} + F_{ub} A_{nt}) = 403.8 kip

ϕR_{nbs} = 302.9 kip

$\phi M_n = \phi R_{nbs}$ (d - t_{fb}) = 440 kip-ft

10. Flange plate compression strength (ϕ = 0.9):

K_c = 0.65 $\quad$ L = s = 3 in

r = t_{fp} / √12 = 0.217 in

K_c L / r = 9 then $R_n = F_{yp}\ t_{fp}$ 7 in = 189 kip

ϕR_n = 170.1 kip

$\phi M_n = \phi R_n$ (d + t_{fp}) = 266 kip-ft

11. Column flange local bending strength (ϕ = 0.9):

t_{fc} = 0.78 in $\quad$ F_{yc} = 50 ksi $\quad$ F_{uc} = 65 ksi

$R_n = 6.25\ F_{yc}\ t_{fc}^2$ = 190.1 kip

ϕR_n = 171.1 kip

$\phi M_n = \phi R_n$ (d + t_{fp}) = 267 kip-ft

12. Column web local yielding strength (ϕ = 1.0):

$l_b = t_{fp}$ = ¾ in $\quad$ k = 1.38 in $\quad$ t_{wc} = 0.485 in

$R_n = F_{yc}\ t_{fc}$ (5k + l_b)= 185.5 kip

ϕR_n = 185.5 kip

$\phi M_n = \phi R_n$ (d + t_{fp}) = 290 kip-ft

13. Column web local crippling strength (ϕ = 0.75):

d_c = 14.2 in $\quad$ Q_f = 1

$$R_n = 0.8t_{wc}^2\left[1 + 3\left(\frac{l_b}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_y t_{fc}}{t_{wc}}}Q_f = 309.7 \text{ kip}$$

ϕR_n = 232.3 kip

$\phi M_n = \phi R_n (d + t_{fp})$ = 363 kip-ft

14. Column web panel zone shear strength: (ϕ = 0.9):

$R_{npz} = 0.6\ F_{yc}\, d_c\, t_{wc}$ = 206.6 kip **ϕR_{npz} = 185.9 kip**

$\phi M_n = \phi R_{npz} (d + t_{fp})$ = 291 kip-ft

15. Web plate bolts shear strength (ϕ = 0.75):

ϕR_{nv} = 24.4 kip

$\phi V_n = 3\ \phi R_{nv}$ = 73 kip

16. Web plate shear strength (ϕ = 0.75):

t_{wp} = ⅜ in

$A_e = t_{wp}$ (9 in - 3(d_h + $^1/_{16}$”))= 2.3 in^2

$V_n = 0.6F_{up}\, A_e$ = 78.3 kip

ϕV_n = 59 kip

17. Web plate weld strength (ϕ = 0.75):

w = ¼” l_w = 9 in

t_e = 0.707 w = 0.177 in

$F_{nw} = 0.6\ F_{E70}$ = 42 ksi

$V_{nw} = F_{nw}\, t_e\, 2l_w$ = 133.6 kip

ϕV_{nw} = 100 kip

18. Column flange rupture (ϕ = 0.75):

$V_{nw} = 0.6F_{uc}\, t_{fc}\, 2l_w$ = 547.6 kip

ϕV_{nw} = 411 kip

19. Web plate bearing strength (ϕ = 0.75):

$R_{nbwp} = 2.4\ d_b\, t_{wp}\, F_{up}$ = 45.7 kip

$\phi V_n = 3\phi R_{nbwp} = 103$ kip

20. Web plate tearout strength ($\phi = 0.75$):

Interior bolt

$s = 3$ in $\quad d_h = d_b + {}^{1}/_{16}" = 0.94$ in $\quad l_c = s - d_h = 2.1$ in

$R_{nto\_i} = 1.2\ l_c\ t_{wp}\ F_{up} = 53.8$ kip $\quad$ **$\phi R_{nto\_i} = 40.4$ kip**

Exterior bolt

$l_c = 1.5$ in $- d_h/2 = 1$ in

$R_{nto\_e} = 1.2\ l_c\ t_{wp}\ F_{up} = 26.9$ kip $\quad$ **$\phi R_{nto\_e} = 20.2$ kip**

$\phi V_n = 2\phi R_{nto\_i} + \phi R_{nto\_e} = 101$ kip

21. Beam web bearing strength ($\phi = 0.75$):

$t_{wb} = 0.355$ in

$R_{nbwb} = 2.4\ d_b\ t_{wb}\ F_{up} = 48.5$ kip $\quad$ **$\phi R_{nbwb} = 36.3$ kip**

$\phi V_n = 3\phi R_{nbwp} = 109$ kip

22. Web plate block shear strength ($\phi = 0.75$):

$l_{gv} = 2s + 1.5$ in $= 7.5$ in $\quad l_{nv} = l_{gv} - 2.5(d_h + {}^{1}/_{16}") = 5$ in

$A_{gv} = l_{gv}\ t_{wp} = 2.8$ in^2 $\quad A_{nv} = l_{nv}\ t_{wp} = 1.9$ in^2

$l_{gt} = 1.5$ in $\quad l_{nt} = l_{gt} - 0.5(d_h + {}^{1}/_{16}") = 1$ in

$A_{gt} = l_{gt}\ t_{wp} = 0.6$ in^2 $\quad A_{nt} = l_{nt}\ t_{wp} = 0.4$ in^2

$U_{bs} = 1$

$V_{nbs} = \min(0.6\ F_{up}\ A_{nv} + F_{up}\ A_{nt};\ 0.6\ F_{yp}\ A_{gv} + F_{up}\ A_{nt}) = 82.5$ kip

$\phi V_{nbs} = 62$ kip