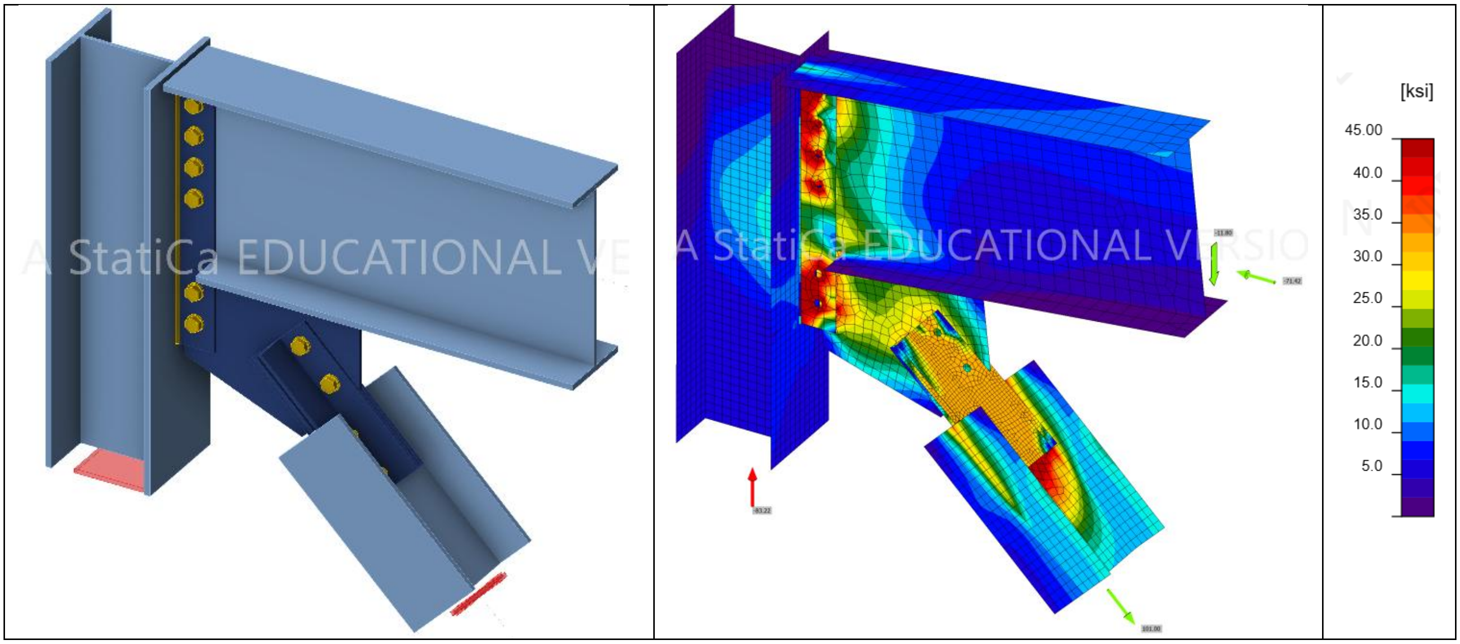
Corner connection with wide flange brace in Ordinary Concentrically Braced Frame (OCBF) System

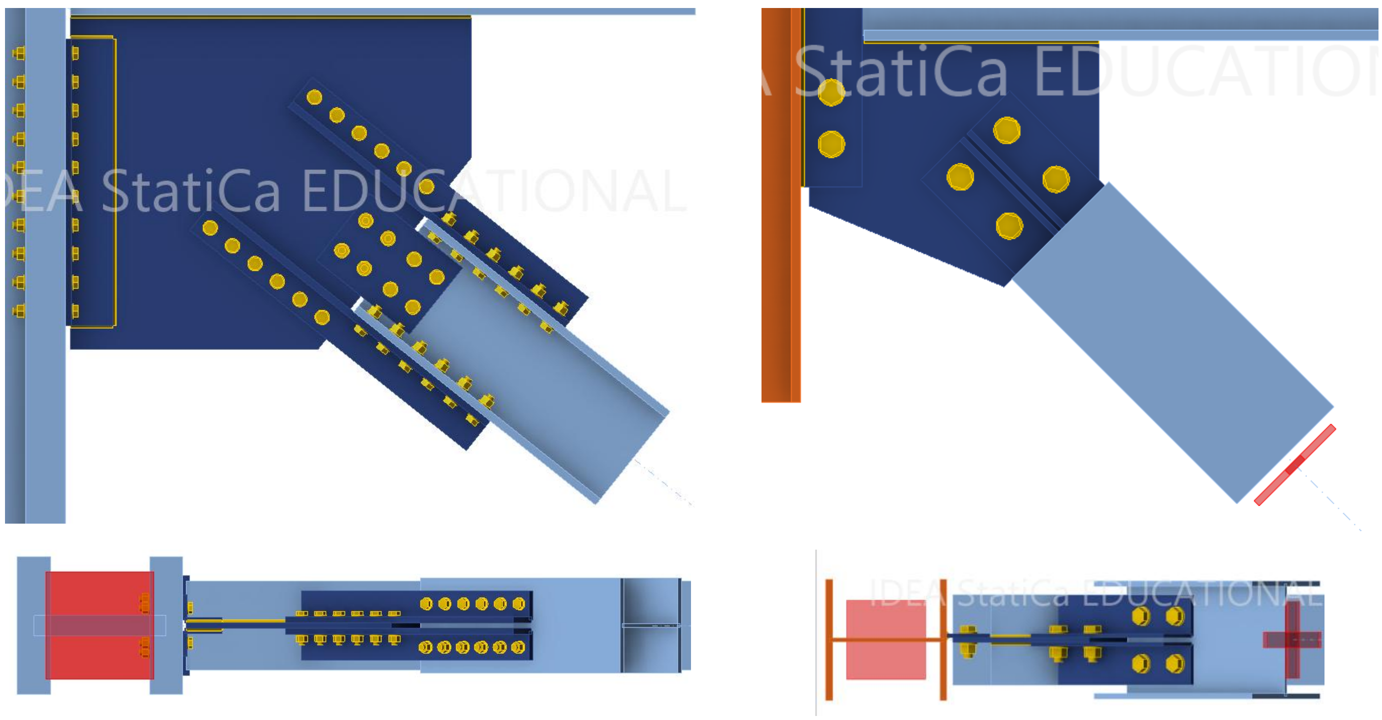


This is the **first verification example** in CBFEM from a **series of seismic vertical brace connections**. It compares a beam-column **corner connection with a wide flange brace member** in an ordinary concentrically braced frame **(OCBF)** system according to a procedure from seismic design manual (AISC 341-16) and CBFEM method in IDEA StatiCa connection.

1. Overview

Vertical brace connections are the critical points of structural stability in steel structures against lateral forces. AISC 360-16 standard serves as a reference, offering methodologies for both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), while AISC 341-16, Seismic Provisions for Structural Steel Buildings, is the standard reference document for the seismic design of steel structures throughout the United States.

In a braced frame, wide flange braced members are quite popular as they are very efficient for axial loads. Two conditions for the brace-to-gusset connection can be considered here as shown in Figure 1, wherein claw angles or double angles are widely preferred connecting elements / stiffening elements.



a) Connection for gusset to the wide flange brace (brace oriented along the plane of gusset plate)

b) Connection for gusset to the wide flange brace (brace oriented perpendicular to the plane of gusset plate)

Figure 1 – Corner connection at beam-column location with wide flange brace member

For the first condition as shown in Figure 1 a), such that the web of brace is oriented along the plane of gusset plate, claw angles connect the top as well as bottom of the brace, whereas shear splice plate connects the brace web to the gusset. For the second condition, such that the top and bottom flanges of the brace are oriented perpendicular to the plane of gusset plate, claw angles are connecting the web of wide flange brace to gusset plate. Connection at beam to gusset plate is a welded connection, while the connection at beam-to-column will depend on the preferences by engineer, erectors and fabricators working on the project.

Refer to https://www.ideastatica.com/support-center/brace-connection-at-beam-column-connection-in-a-braced-frame-aisc for CBFEM evaluation of wide flange brace connection in non-seismic zones.

2. Problem Description

The objective of this example is to verify the component-based finite element method (CBFEM) for a wide flange braced connection in an Ordinary Concentrically Braced Frame (OCBF) of one-story steel building as shown in Figures 2 and 3. The results obtained from calculation method which are based on AISC 360-16 Specifications and AISC 341-16 are compared with results obtained from the CBFEM analysis using IDEA StatiCa software version 23.0.

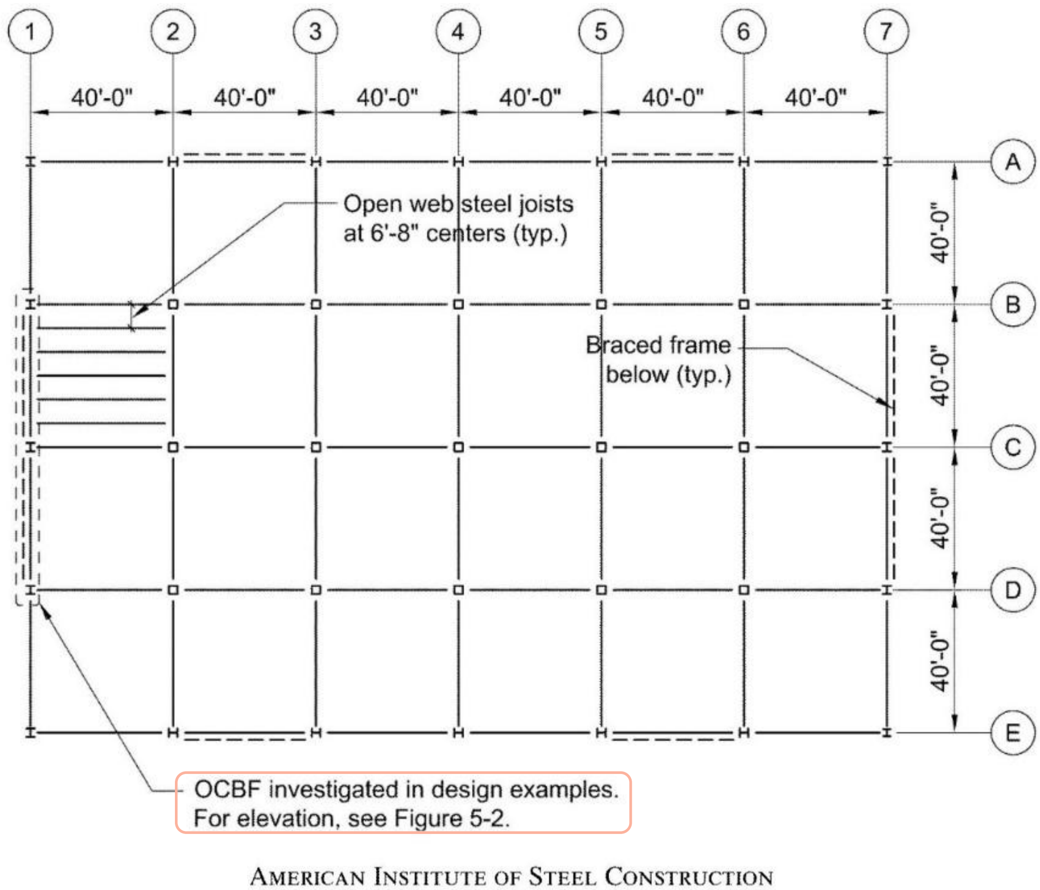


Figure 2 - Floor plan of steel building highlighting the investigated OCBF (AISC 341, 2016)

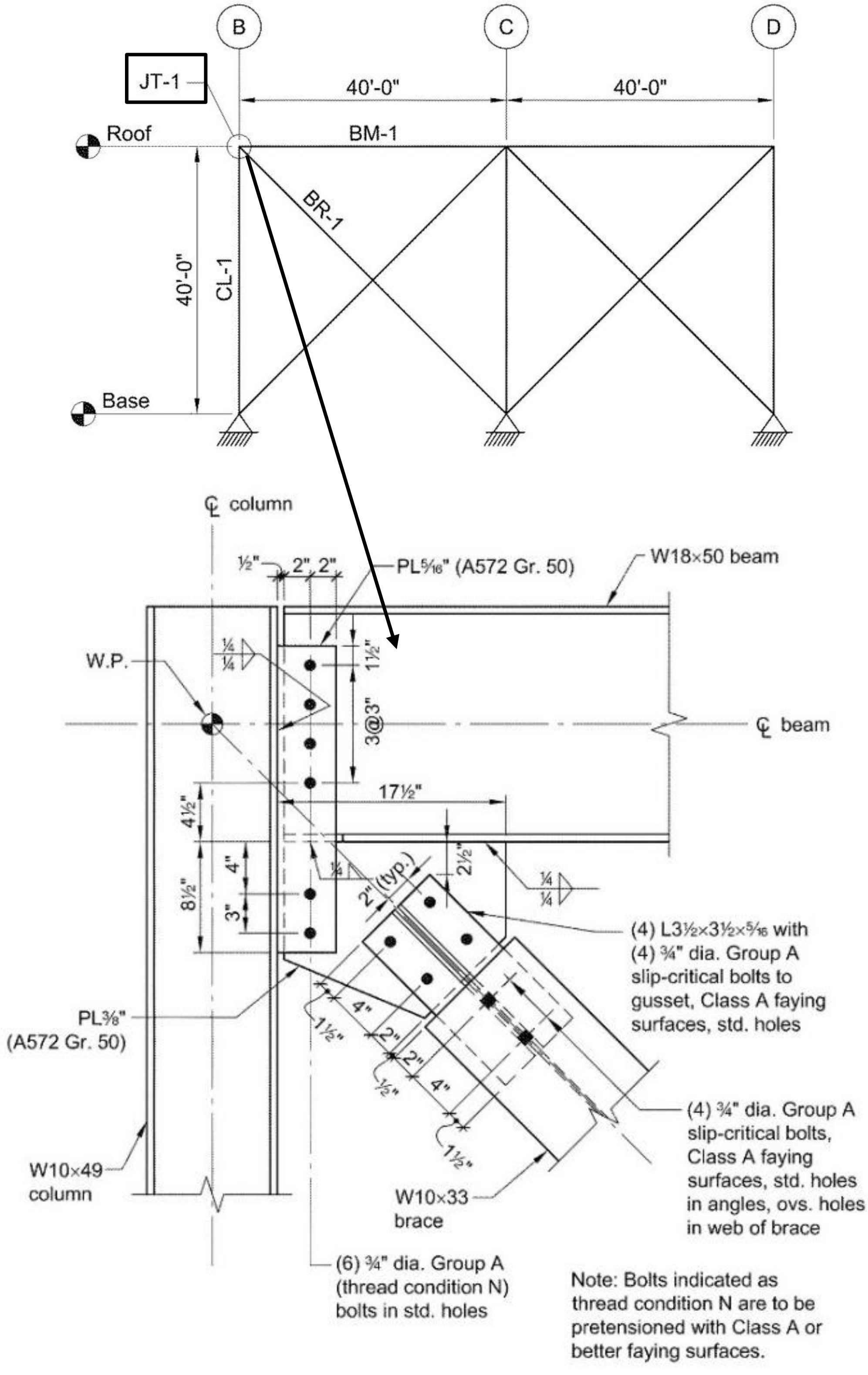


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Figure 3 - Elevation of steel building and considered corner connection details (AISC 341, 2016)

A single-plate connection (fin plate) is used to connect the beam and gusset to the column and a welded connection between the beam and gusset plate is provided. Four claw angles are used to connect the wide flange brace to the gusset plate via bolted connection with two ¾ in. diameter Group A Slip-critical bolts in double shear, Class A faying surface as shown in Figure 3. The **bolt holes in the web of the brace are oversize** owing to the erection tolerances, since the claw angles are connected only to the web of the brace and not to the flange. Hence, accounting for the reduction in effective area due to shear lag in brace web.

The details for the members and connection of the presented connection are as follows:

Member Details

1. **Beam cross-section**

- W18x50
- ASTM A992

2. **Column cross-section**

- W10x49
- ASTM A992

3. **Brace cross-section**

- W10x33
- ASTM A992

Plate Details

1. **Gusset Plate**

- 3/8" thickness
- ASTM A572-50

2. **Angle Sections** (connecting brace to gusset plate)

- (4) L 3-1/2x3-1/2x5/16
- ASTM A36

3. **Fin Plate** (connecting beam web to column flange)

- 5/16" thickness
- ASTM A572-50

Connection Details

1. **Welds**

- E70xx electrode
- 1/4" double sided fillet weld at fin plate to column flange.
- 1/4" double sided fillet weld at gusset plate to beam bottom flange.

2. **Bolts**

- (4) 3/4" diameter Group A (N) bolts in standard holes on fin plate connecting to beam web.
- (2) 3/4" diameter Group A (N) bolts in standard holes on fin plate connecting to gusset plate.
- (4) 3/4" diameter Group A (N) slip-critical bolts on angles connecting to gusset plate; standard holes.
- (4) 3/4" diameter Group A (N) slip-critical bolts on angles connecting to wide flange brace web; standard holes in angle; oversized holes in web of brace.

3. Modelling and Analysis of steel connection in IDEA StatiCa

The given one-story steel building was modelled in **SAP2000** software as shown in Figure 4.

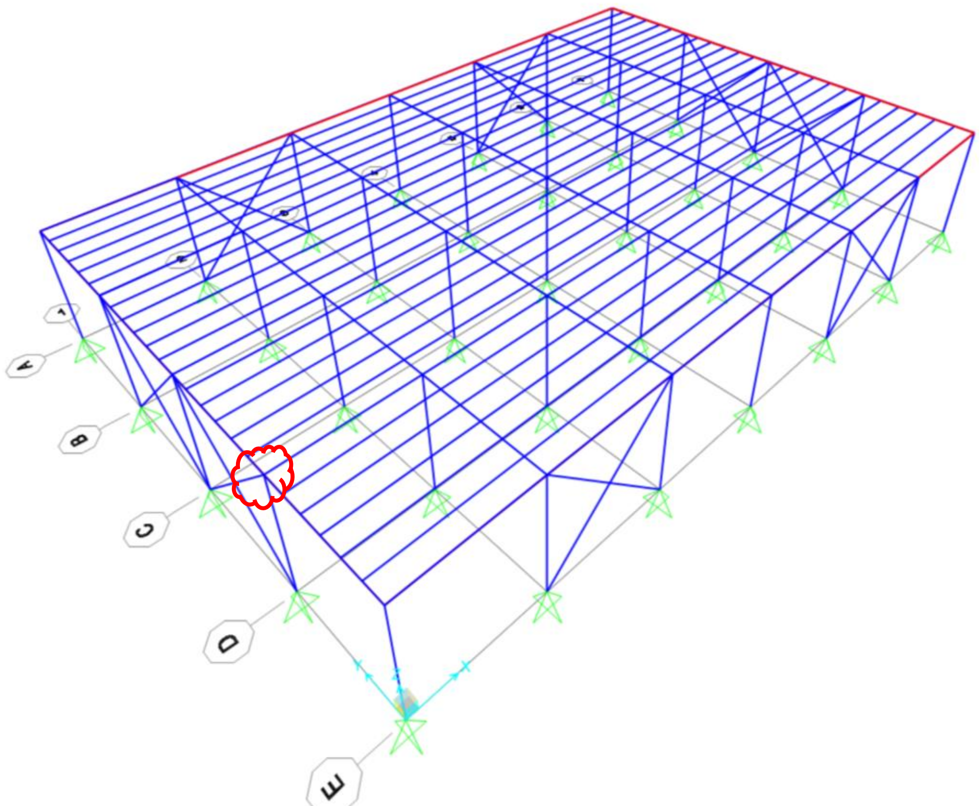

Figure 4 - 3D view of SAP2000 model for steel building

Gravity loads were calculated and then applied to the building frame. The seismic lateral forces were applied at the level as point loads representing the full seismic load for considered side of building. Base reactions for the frame were checked by hand calculations using the simple statics by considering the overturning of the frame. Seismic load combinations as prescribed by ASCE7-16 were used and the analysis and design of building was performed in SAP2000.

E = f (E_v; E_h) (defined in Section 12.4.2 or 12.4.3.1) is combined with the effects of other loads, according to ASCE 7-16 Section 2.3.1, basic combinations below are considered.

- $1.2D + E_v + E_h + L + 0.2S$
- $0.9D - E_v + E_h$

where the seismic load effect with overstrength, E_m = f (E_v; E_{mh}), defined in Section 12.4.3, is combined with the effects of other loads, the following seismic load combination for structures is applied:

- $1.2D + E_v + E_{mh} + L + 0.2S$
- $0.9D - E_v + E_{mh}$

where,

D = dead load

E_m = combined applied design force on the horizontal and vertical seismic load effects

E_{mh}= The effect of horizontal seismic forces, including overstrength

L = Live load

L_r = roof live load

S = snow load

W = wind load

E_h = horizontal seismic load effect

E_v = vertical seismic load effect

With oversize holes in the diagonal brace, the required strength for the bolt slip need not exceed the load effects calculated using the seismic load combinations without the overstrength factor.

To account for the oversize holes in the web of brace, the default resistance factor of 1 for standard holes was modified to 0.85 for the slip-resistant joint in the code setup of the IDEA StatiCa software, as shown in Figure 5.

High-Strength Bolts in Slip-Critical Connections

Slip-critical connections shall be designed to prevent *slip* and for the *limit states* of *bearing-type connections*. When slip-critical bolts pass through *fillers*, all surfaces subject to slip shall be prepared to achieve design slip resistance.

The available slip resistance for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s \tag{J3-4}$$

(a) For standard size and short-slotted holes perpendicular to the direction of the *load*

$\phi = 1.00$ (LRFD) $\quad \Omega = 1.50$ (ASD)

(b) For oversized and short-slotted holes parallel to the direction of the load

$\phi = 0.85$ (LRFD) $\quad \Omega = 1.76$ (ASD)

(c) For long-slotted holes

$\phi = 0.70$ (LRFD) $\quad \Omega = 2.14$ (ASD)

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Code and calculation settings

LRFD - Resistance factors ϕ

Tensile and shear strength - bolts	0.75
Combined tensile and shear strength - bolts	0.75
Bearing at bolt holes	0.75
Fillet welds	0.75
Material resistance factor	0.9
Slip resistant joint	0.85
Strength reduction factor for anchors in tension	0.7
Strength reduction factor for anchors in shear	0.65

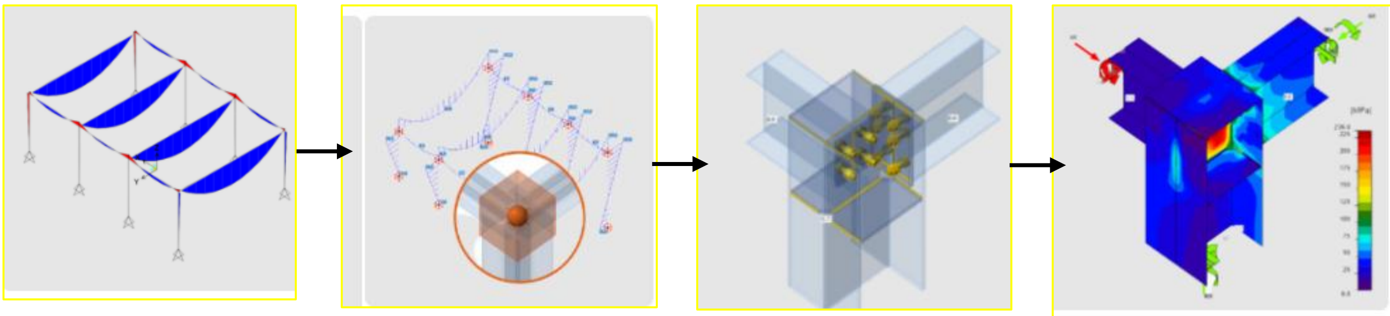
IDEA StatiCa®

Figure 5 - Modification of resistance factor in CBFEM for consideration of OVS holes

To perform the CBFEM analysis of the given connection JT-1, it was imported into IDEA StatiCa Version 23.0 software using in-built software links after the analysis and design was complete in SAP2000.

The connection imported in IDEA StatiCa was checked for the given connection details, and the **base model** was prepared. The fin plate and gusset plate are modelled using "stiffening plate" operation, and the angle sections are modelled as stiffening members in CBFEM. A typical workflow which was followed is highlighted in Figure 6. The column is assigned to be a bearing member in CBFEM, such that the loads on it are balanced by "loads in equilibrium".

Similarly, connection and members from most of the FEA/ CAE and CAD programs available in current market can be easily exported into IDEA StatiCa using BIM links. More details can be found in https://www.ideastatica.com/bim.



a) Perform global analysis in SAP2000

b) Import the joint from SAP2000 to IDEA StatiCa Connection

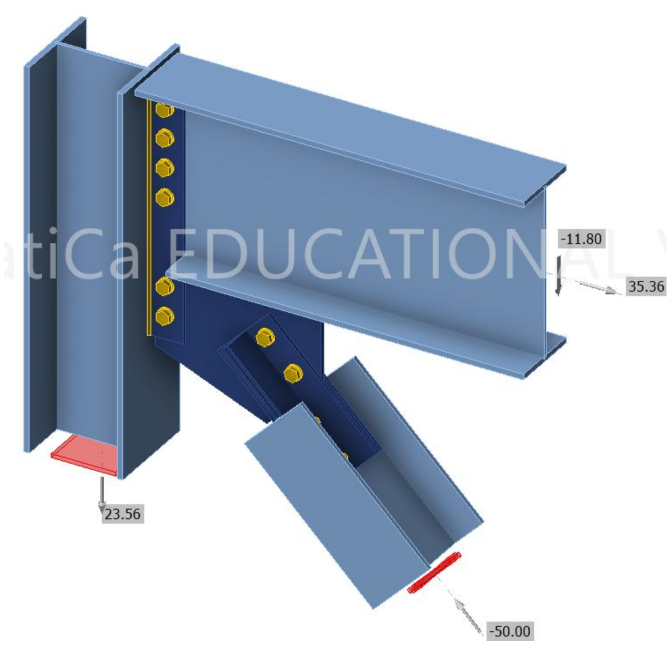
c) Design selected connection in IDEA StatiCa

d) Perform code-check of connection and optimize connection (if required)

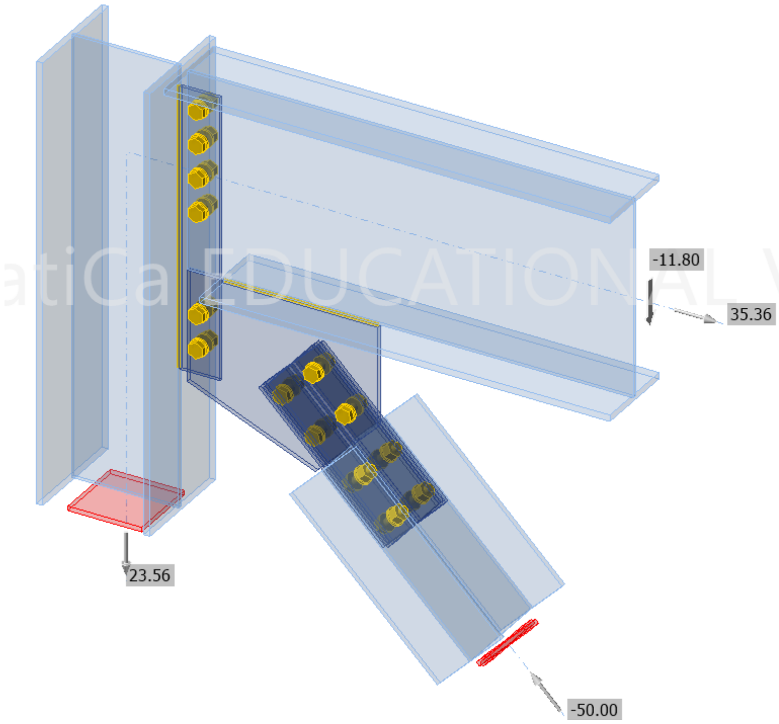
Figure 6- Workflow of connection from SAP2000 to IDEA StatiCa (IDEA StatiCa, n.d.)

IDEA StatiCa offers three views for the connection model – **Solid, Transparent and wireframe view** as shown in Figure 7, which provides enhanced depth and realism in model visualization. Solid model is the default view of the connection in IDEA StatiCa. Transparent view is helpful in case of complex joints for efficient positioning of several connecting elements. While wireframe view provides valuable insights about the loads at the node. It is essential to use "Loads in Equilibrium" option in CBFEM when balancing the unbalanced force in joint.

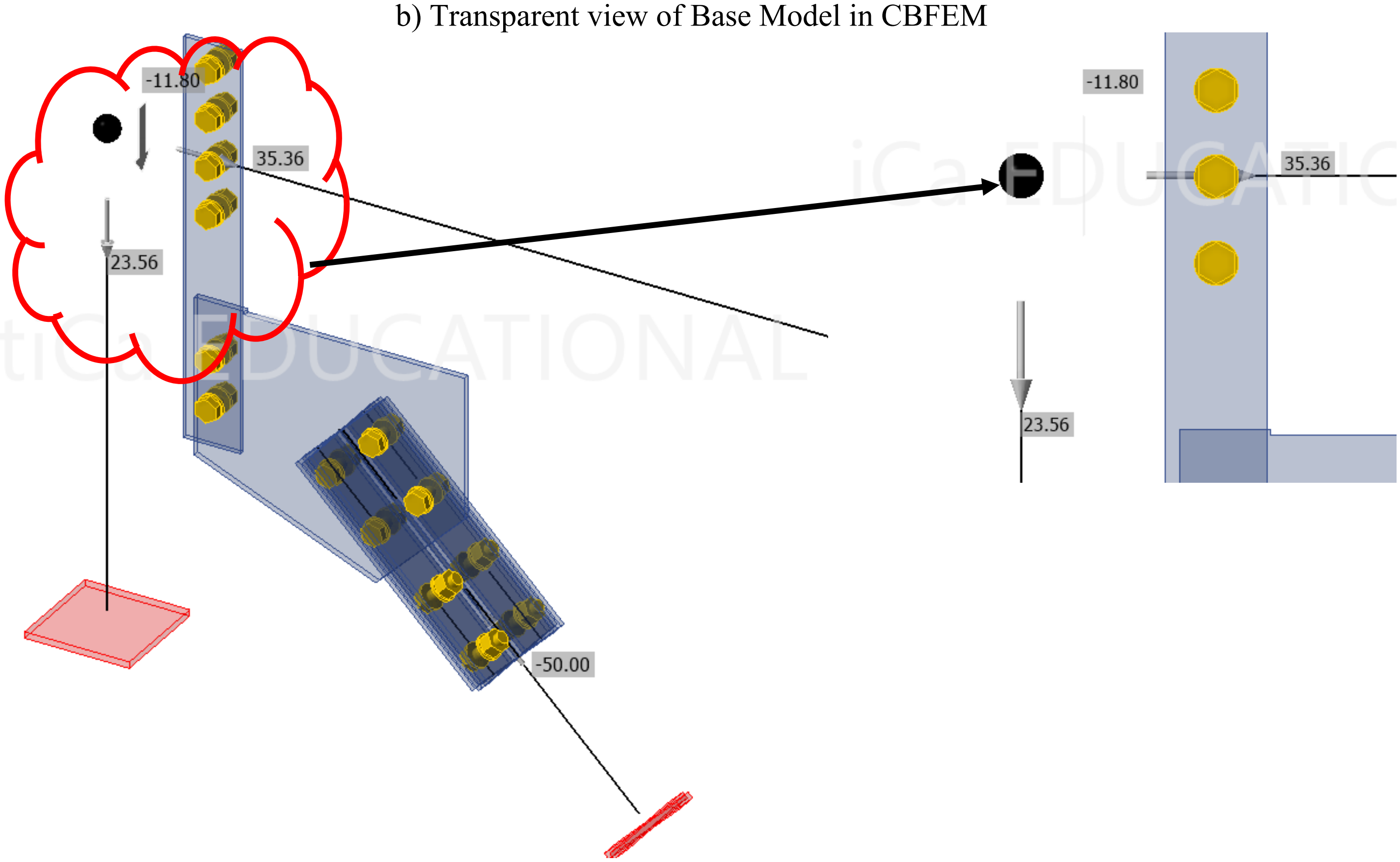
Beam and column are assigned **N-V_y-V_z-M_x-M_y-M_z (Fixed)** as the **model type**, such that the forces are in the node. Whereas the wide flange **brace member** is assigned **N-V_y-V_z (Pinned)** as **model type**, such that the forces are in node.



a) Solid view of Base Model in CBFEM



b) Transparent view of Base Model in CBFEM



c) Wireframe view of Base Model in CBFEM

Figure 7 - Configurations of the view of the base model for OCBF in CBFEM – Solid, Transparent and Wireframe View.

4. Verification of Resistance in CBFEM – Joint Design Resistance (DR)

Joint Design Resistance (DR) analysis type in CBFEM helps to estimate reserve in the connection resistance based on the plastic strains and von-Mises stresses for the action of loads. Once the design loads are assigned, the software automatically proportionally increases all load components until one of the included checks does not satisfy.

Design Resistance analysis conducts check for the following components: Plastic strain in plates, Bolts – shear, tension, and a combination of tension and shear, and welds.

Joint Design Resistance analysis was performed for both – tensile loads and compression loads individually, and the reserve in the connection resistance was estimated, before we began to evaluate the connection for design loads and several limit states in detail further. The loads were gradually increased in brace member starting from 50kips (with increments of 5kips each) until any of the following is achieved: 5% of plastic strain in plates or 100% strength capacity in bolts or 100% strength capacity in welds. The results for the obtained resistance factor value from CBFEM vs the applied axial load is shown in Figure 8.

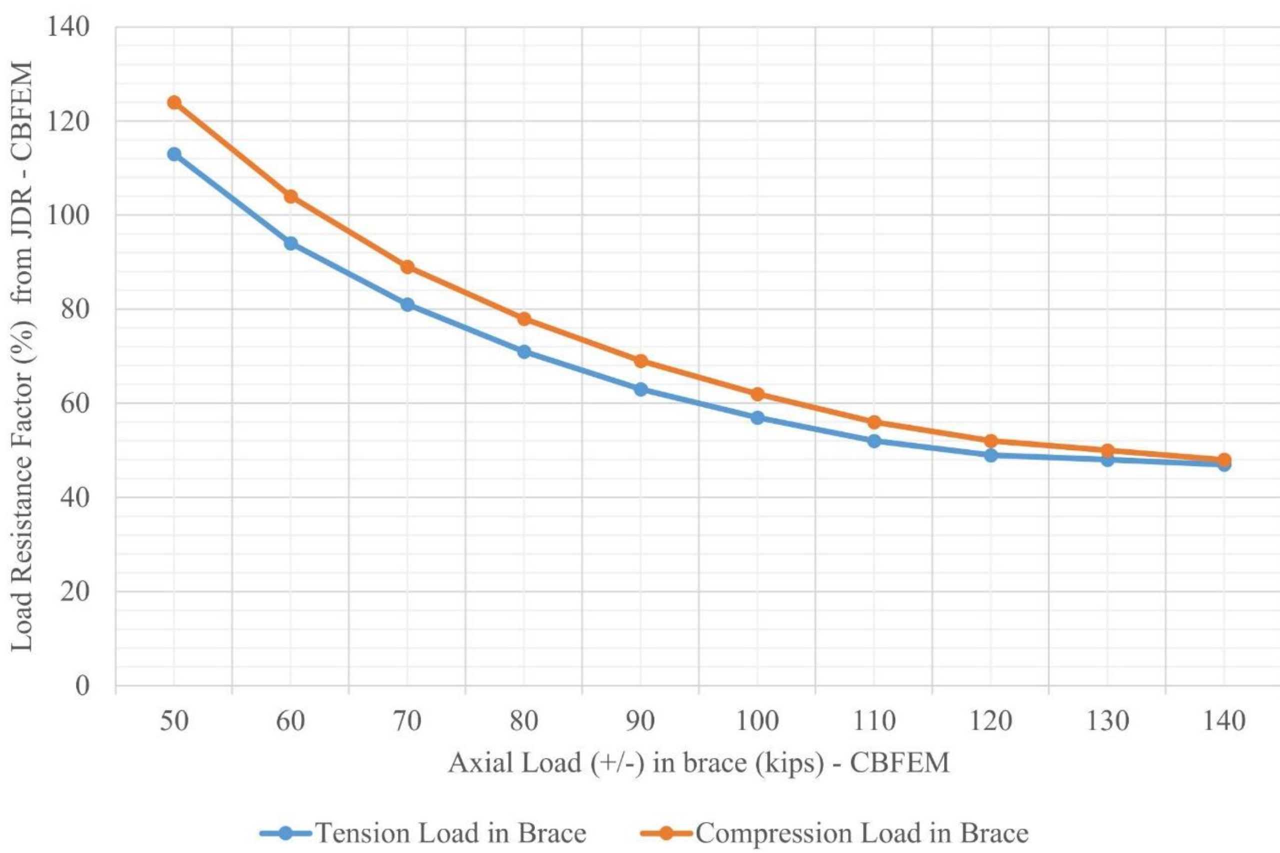


Figure 8 – Curve of load resistance factor from JDR vs axial load in brace for the corner connection - CBFEM

For the case of axial tensile load in brace, the plastic strain and von Mises stresses are obtained for three loads from the Joint Design Resistance (DR) is shown in Figure 10 along with their deformed shapes, considering a deformed shape factor of 1 in CBFEM. The 100% joint design resistance was found for the case of axial tensile load of 57 kips in the brace. For this axial load, the slip-critical bolts in the brace members reached their 100% utilization. CBFEM provides the users with the ratio of maximal load to the design load in the form of a simple diagram in the Joint Design Resistance analysis. This is shown in Figure 9 for the case of the action of the axial tension load of 57 kips, such that the design resistance is approximately close to the design axial tension load.

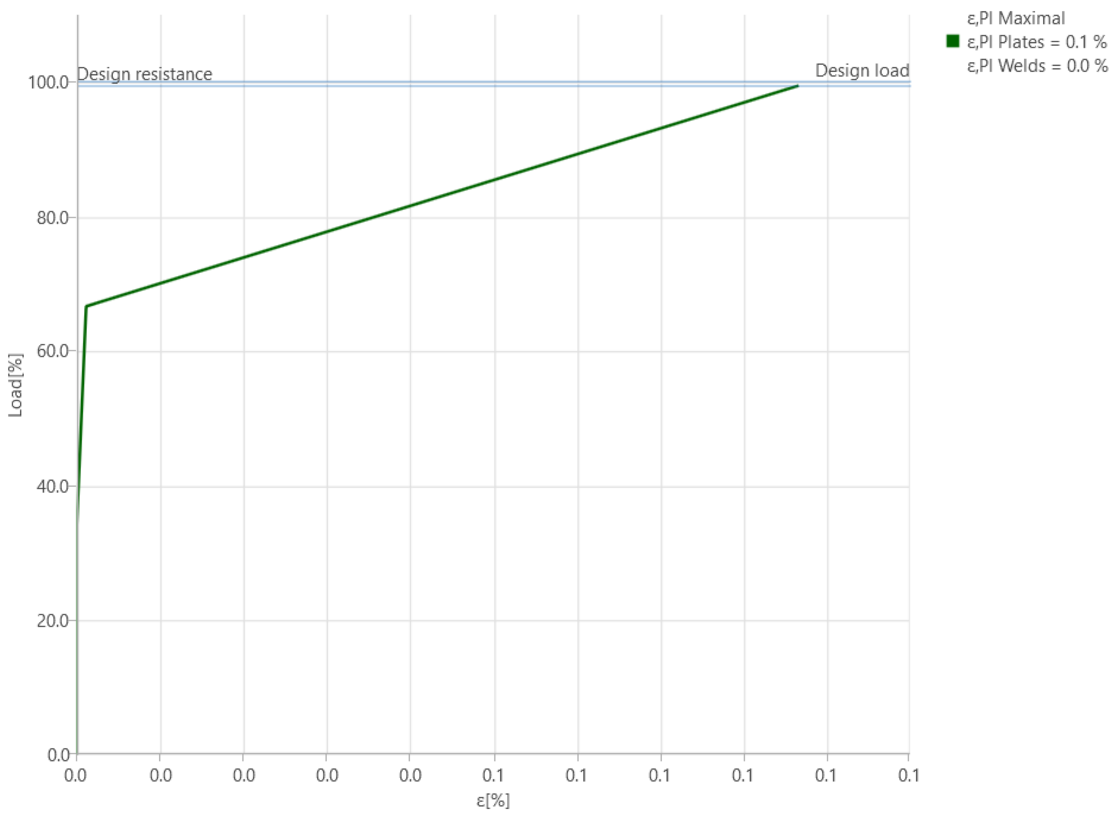
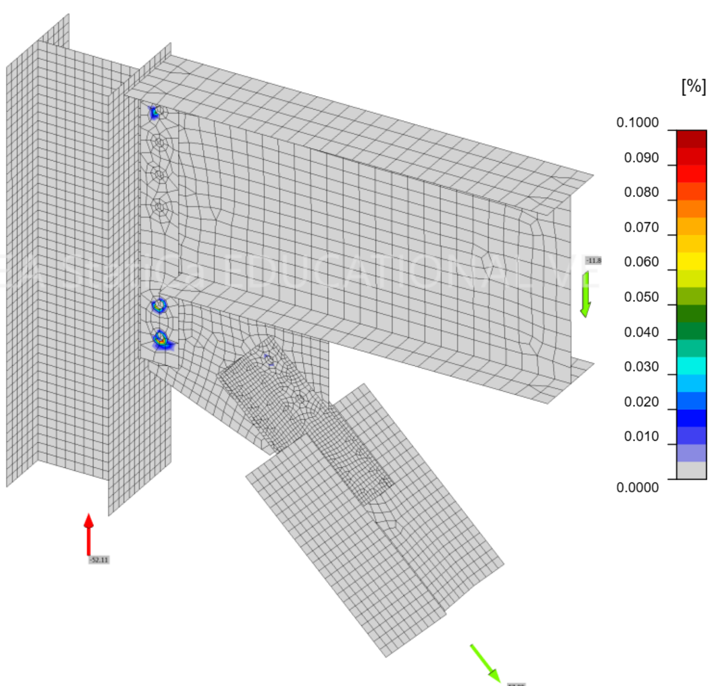
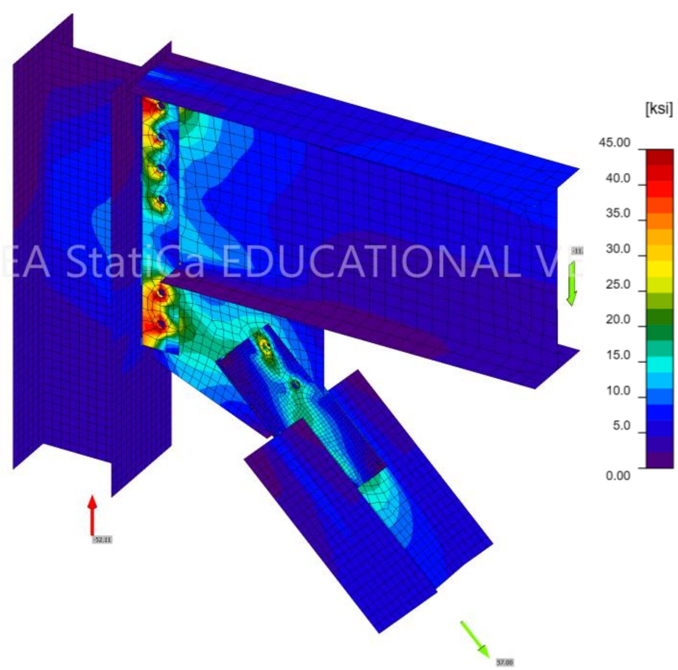


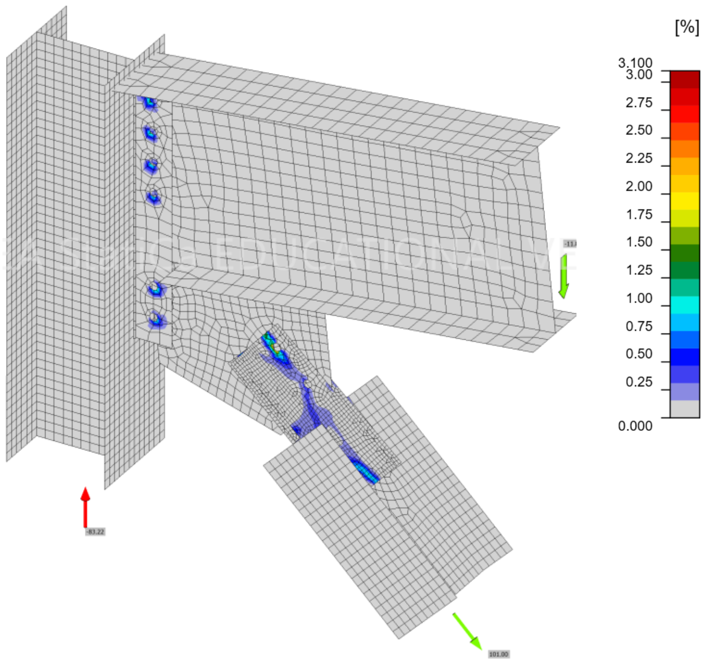
Figure 9 - Ratio of maximal load to the design load for axial tensile load of 57 kips in Joint Design Resistance analysis - CBFEM



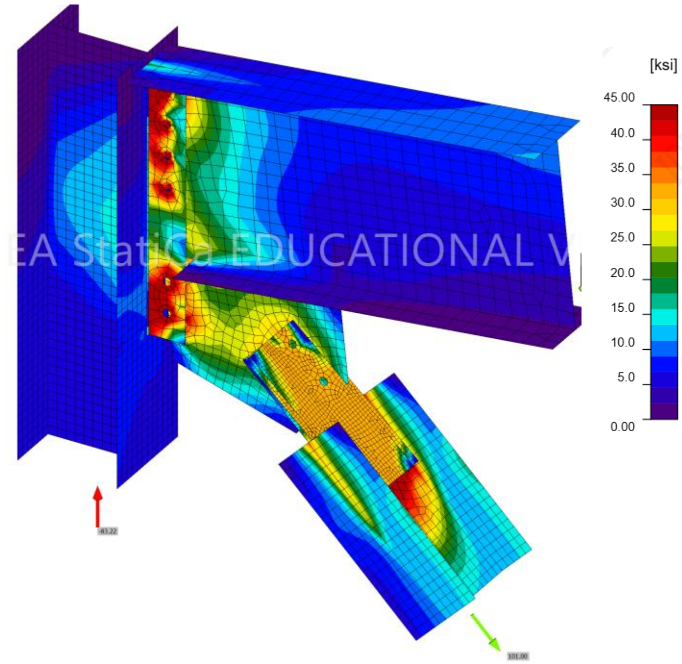
a) Plastic strains for $P_{tension}$ = 57 kips
(Deformed shape factor = 1)



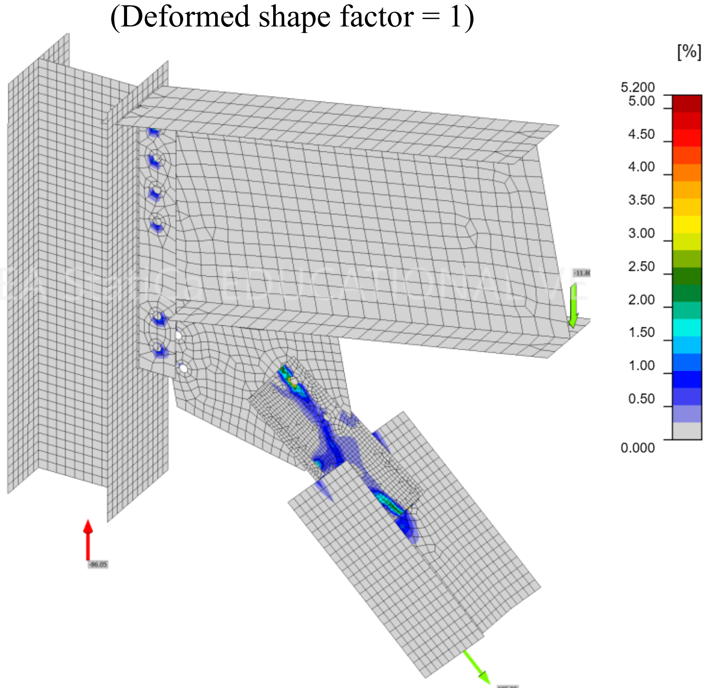
a) von Mises stresses for $P_{tension}$ = 57 kips
(Deformed shape factor = 1)



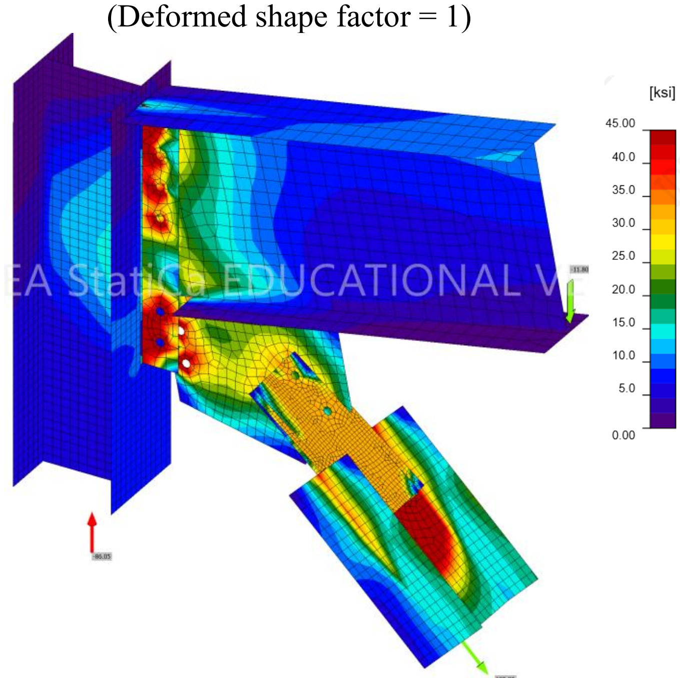
b) Plastic strains for $P_{tension}$ = 101 kips
(Deformed shape factor = 1)



b) von Mises stresses for $P_{tension}$ = 101 kips
(Deformed shape factor = 1)



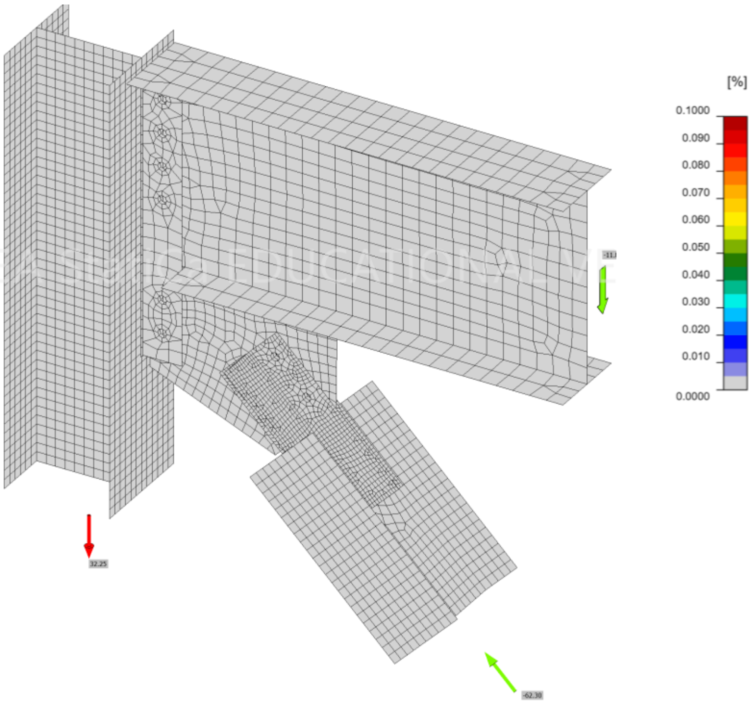
c) Plastic strains for $P_{tension}$ = 105 kips
(Deformed shape factor = 1)



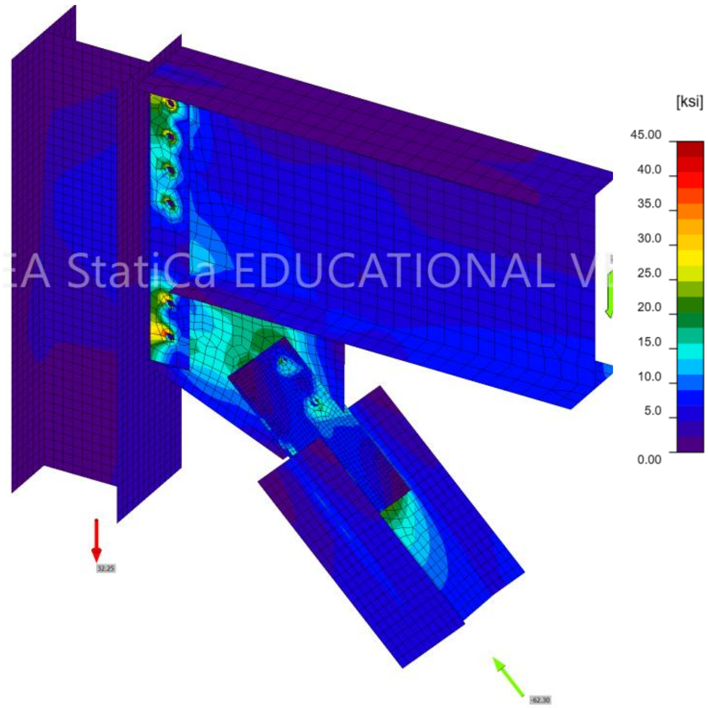
c) von Mises stresses for $P_{tension}$ = 105 kips
(Deformed shape factor = 1)

Figure 10 - Joint Design Resistance (DR) of corner connection for axial tensile load in brace - CBFEM

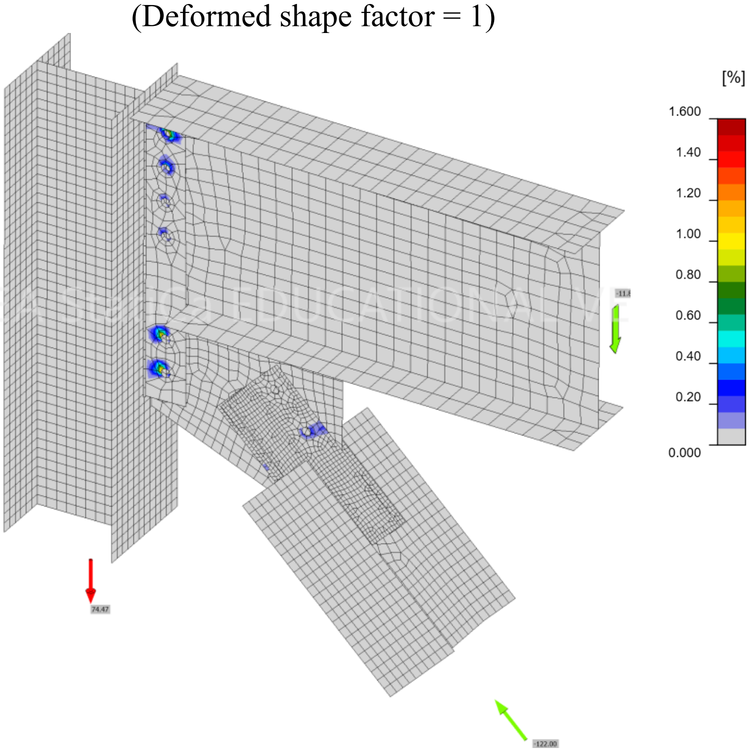
Further, for an axial tensile load of 101 kips, the 100% utilization of bearing bolts in fin plates is reached and at 105 kips as shown in Figure 10, 5% plastic strain in the angle sections connecting the brace web to gusset plate is reached in CBFEM.



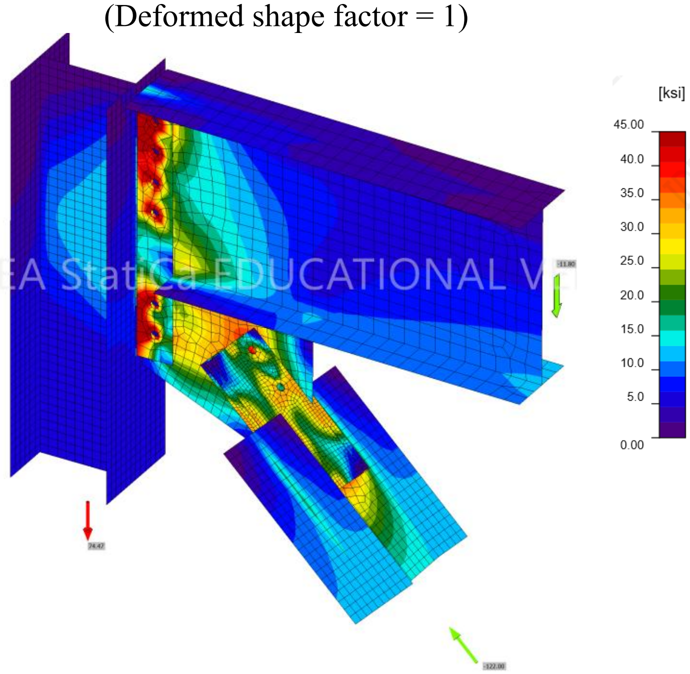
a) Plastic strains for $P_{compression}$ = 62.3kips
(Deformed shape factor = 1)



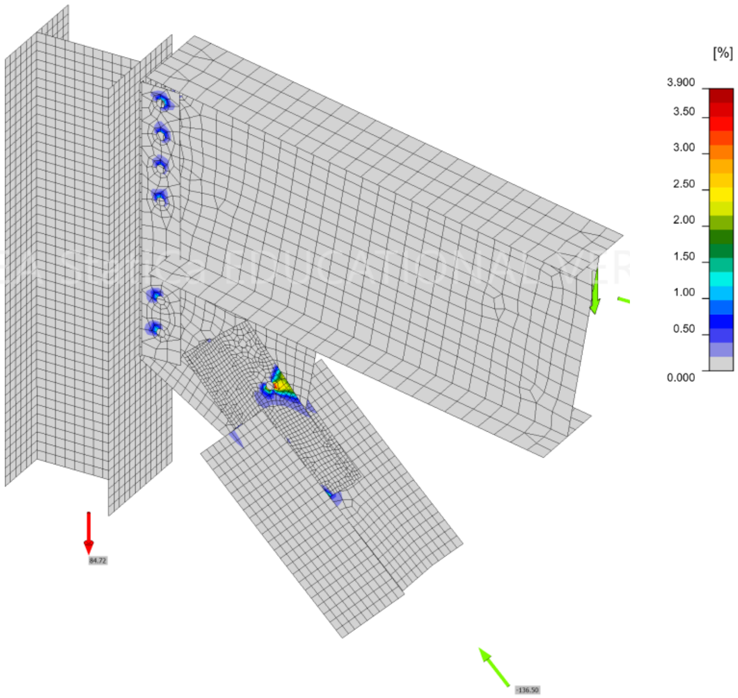
a) von Mises stresses for $P_{compression}$ = 62.3kips
(Deformed shape factor = 1)



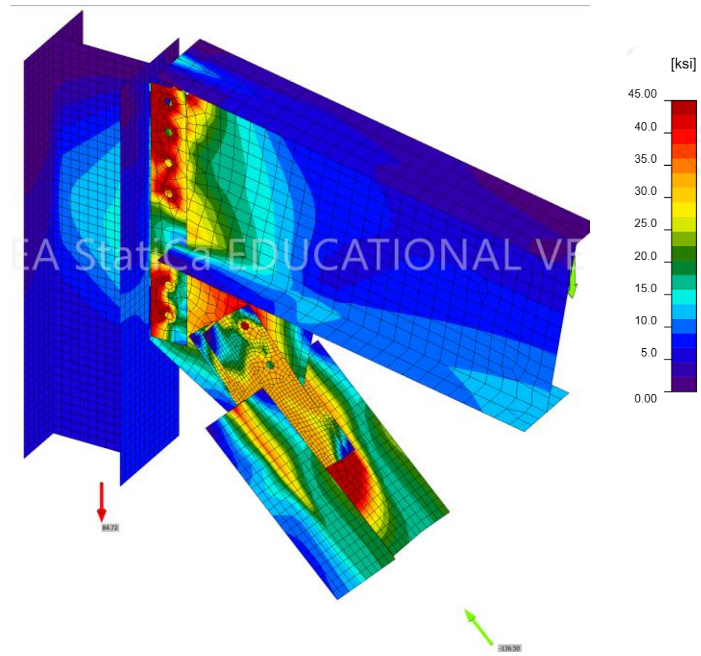
b) Plastic strains for $P_{compression}$ = 122kips
(Deformed shape factor = 1)



b) von Mises stresses for $P_{compression}$ = 122kips
(Deformed shape factor = 1)



c) Plastic strains for $P_{compression}$ = 136.5kips
(Deformed shape factor = 1)

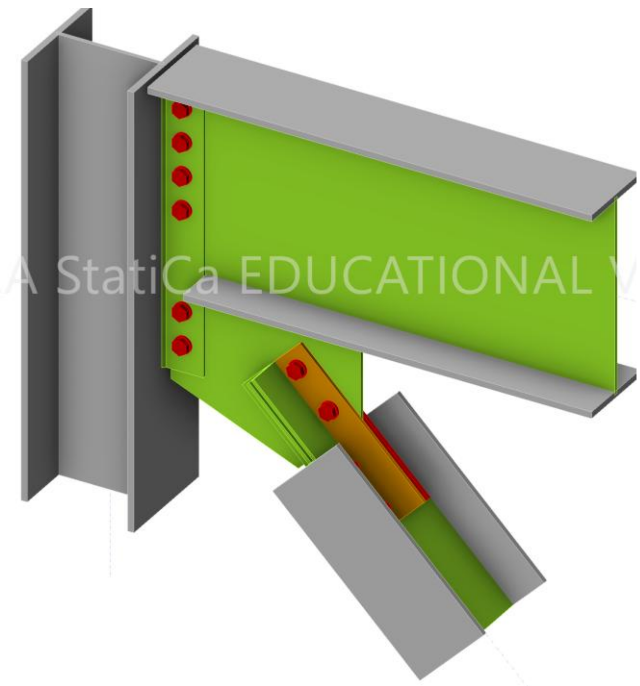


c) von Mises stresses for $P_{compression}$ = 136.5kips
(Deformed shape factor = 1)

Figure 11 - Joint Design Resistance (DR) of corner connection for axial compression load in brace - CBFEM

The design resistance for the connection when subjected to the axial compression loads in the brace, such that loads in other members are balanced by "loads in equilibrium" as shown in Figure 11. The design resistance here is observed to be higher than that for the case of axial tension loads by an average of 23% as per CBFEM from JDR analysis. For the load of 62.3 kips, 100% in slip-critical bolts of brace members is reached, followed by 100% utilization of bearing-type bolts in fin plate at 122 kips of axial load and 5% plastic strain in plate at 136.5 kips load as shown in Figure 10 along with their deformed shapes for a deformed shape factor of 1.5 in CBFEM.

Visualization of connection or member failure can be easily predicted in CBFEM using the "Traffic light type format" of overall check as obtained from the CBFEM after running the analysis. Additional, details are as given in the Figure 12.



Visualization of the overall check in CBFEM (Traffic light type format)

1. **GREY** – Represents those elements which, although could be working well, might be beneath the optimum range (**60%**) of percentage utilization, and therefore will be safe.
2. **GREEN** - Represents the elements, which have a utilization percentage between **60% to 95%.**
3. **ORANGE** – Represents components, that despite not exceeding the maximum capacity, are working at the limit, i.e. **between 95% and 100%.**
4. **RED** – Represents the components that does not satisfy checks and have a utilization **beyond 100%.**

Figure 12 - Visualization of failure in connection and member in overall check of CBFEM

Based on the observation for the given connection from Figure 12, conclusions are tabulated in Table 1,

Sr No	**Color**	**Elements**	**Comments**
1	**GREY**	Column, Beam Flanges, Brace Flanges	Elements are safe for action of loads.
2	**GREEN**	Beam web, Fin Plate, Gusset Plate, Brace web, Weld connecting fin plate to column.	Elements are safe for action of loads.
3	**ORANGE**	Double angles connecting brace web to gusset plate.	Elements are on verge of failure, for the applied loads.
4	**RED**	Some double angles connecting brace web to gusset plate, bolt connections.	Elements are failing for the applied loads; utilization is beyond 100%.

Table 1 – Summary for "Traffic light type format" evaluation of corner connection in CBFEM

The summarized results of joint design resistance (DR) for the given connection in visual graphical form is shown in Figure 13.

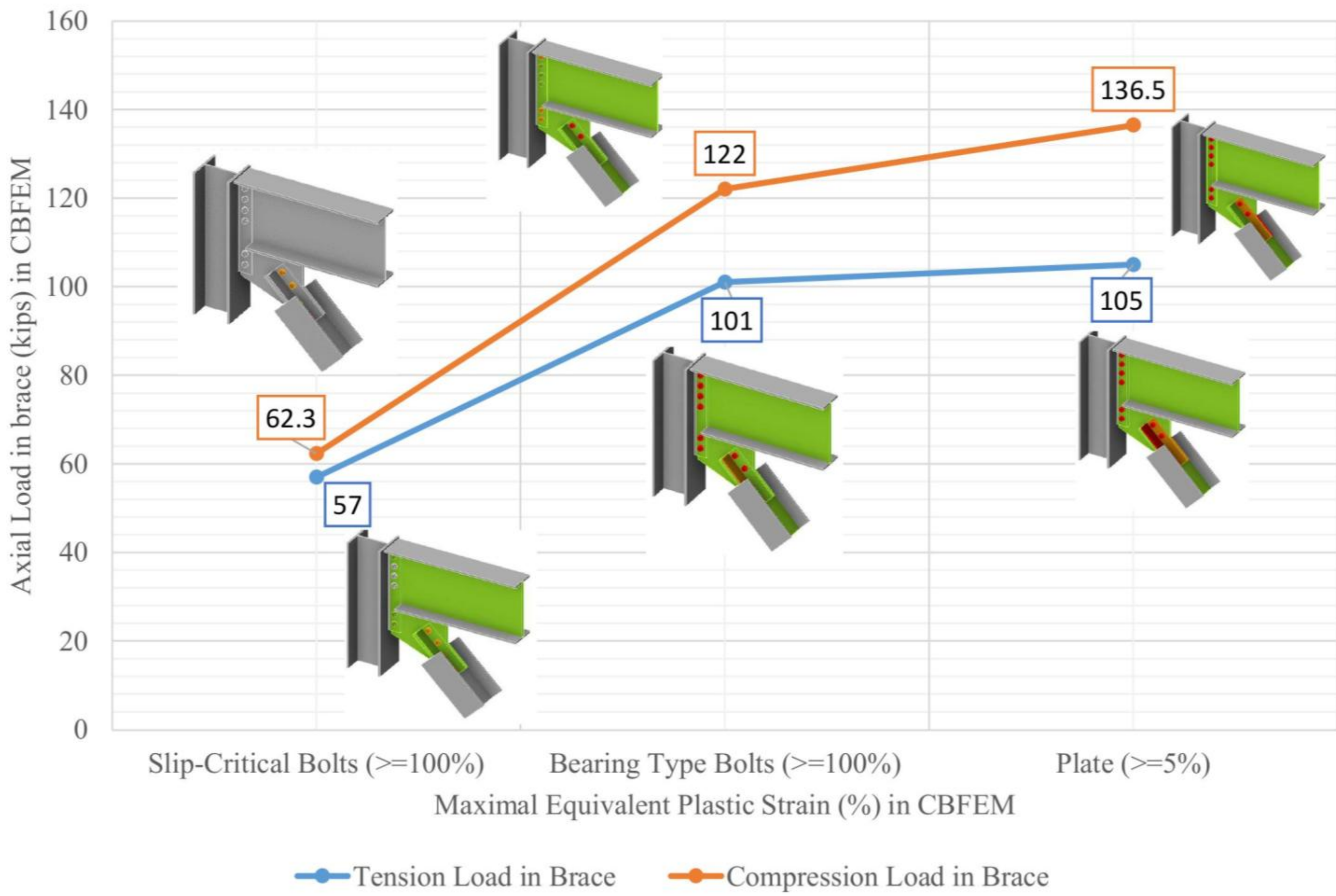


Figure 13 - Summary of Joint Design Resistance (DR) for corner connection with respect to the overall check in CBFEM

5. Code check of connection for design loads in CBFEM

The stress-strain analysis of the given connection is performed for the action of design loads in the connecting members such that “loads in equilibrium” is followed in CBFEM. Detailed calculation for the design loads of the presented connection can be found in Appendix A. The given connection is found to be safe for the action of both design loads – tension and compression, as shown in Figure 14 and Figure 15 respectively.

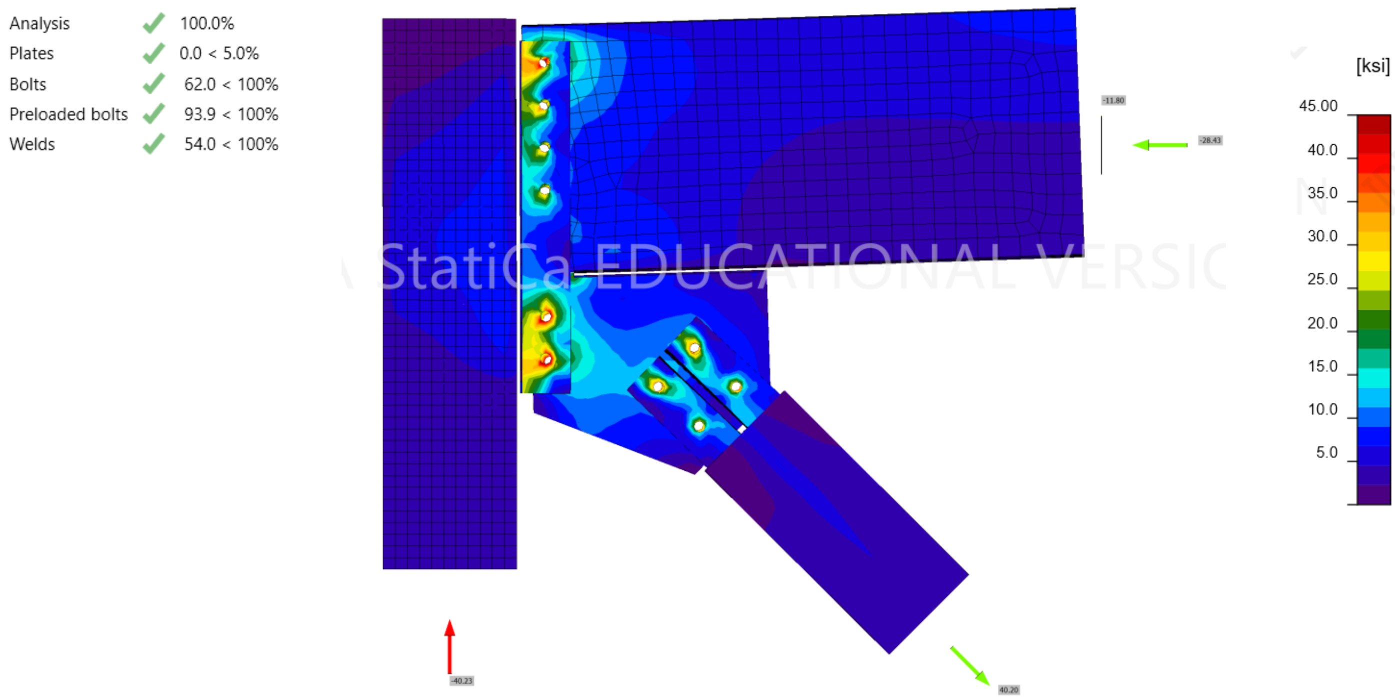


Figure 14 - von Mises stresses in connection for the action of design axial tension load in brace - CBFEM

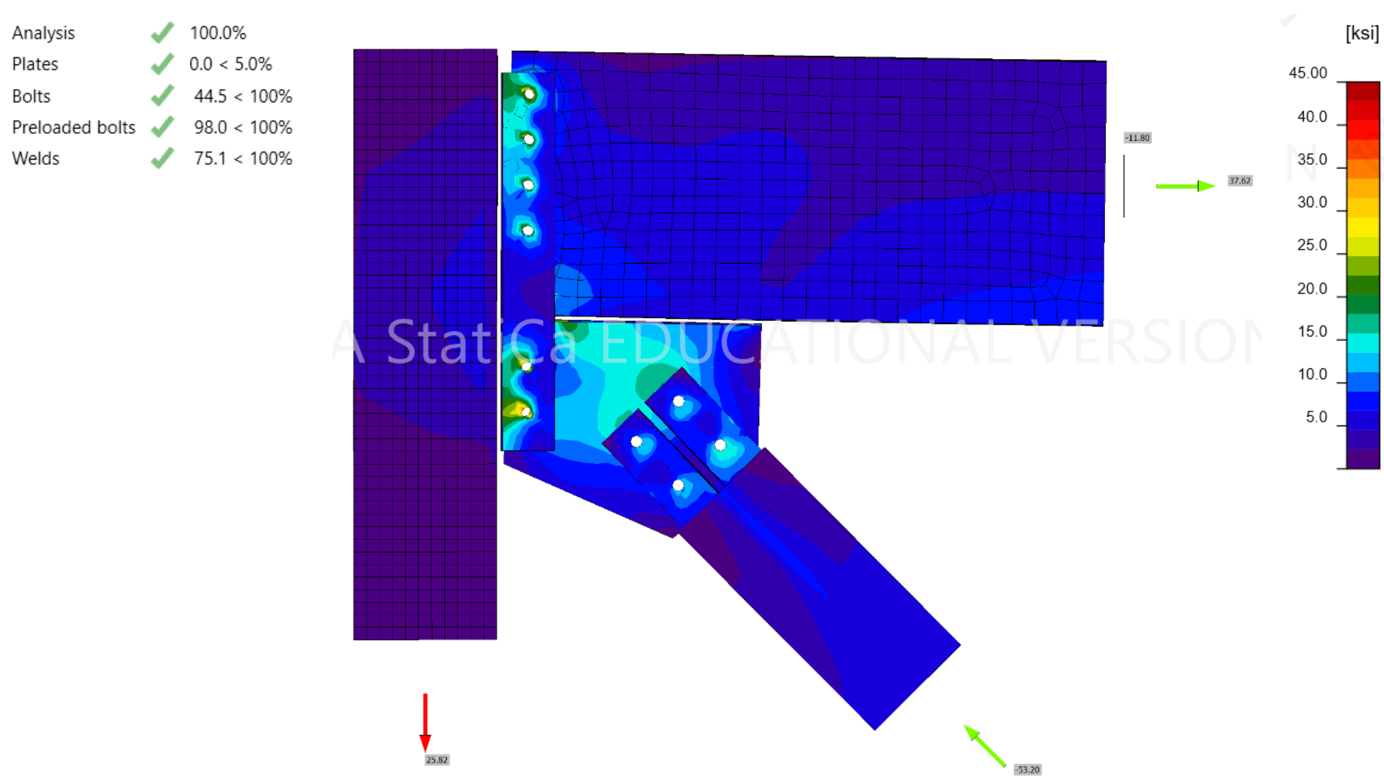


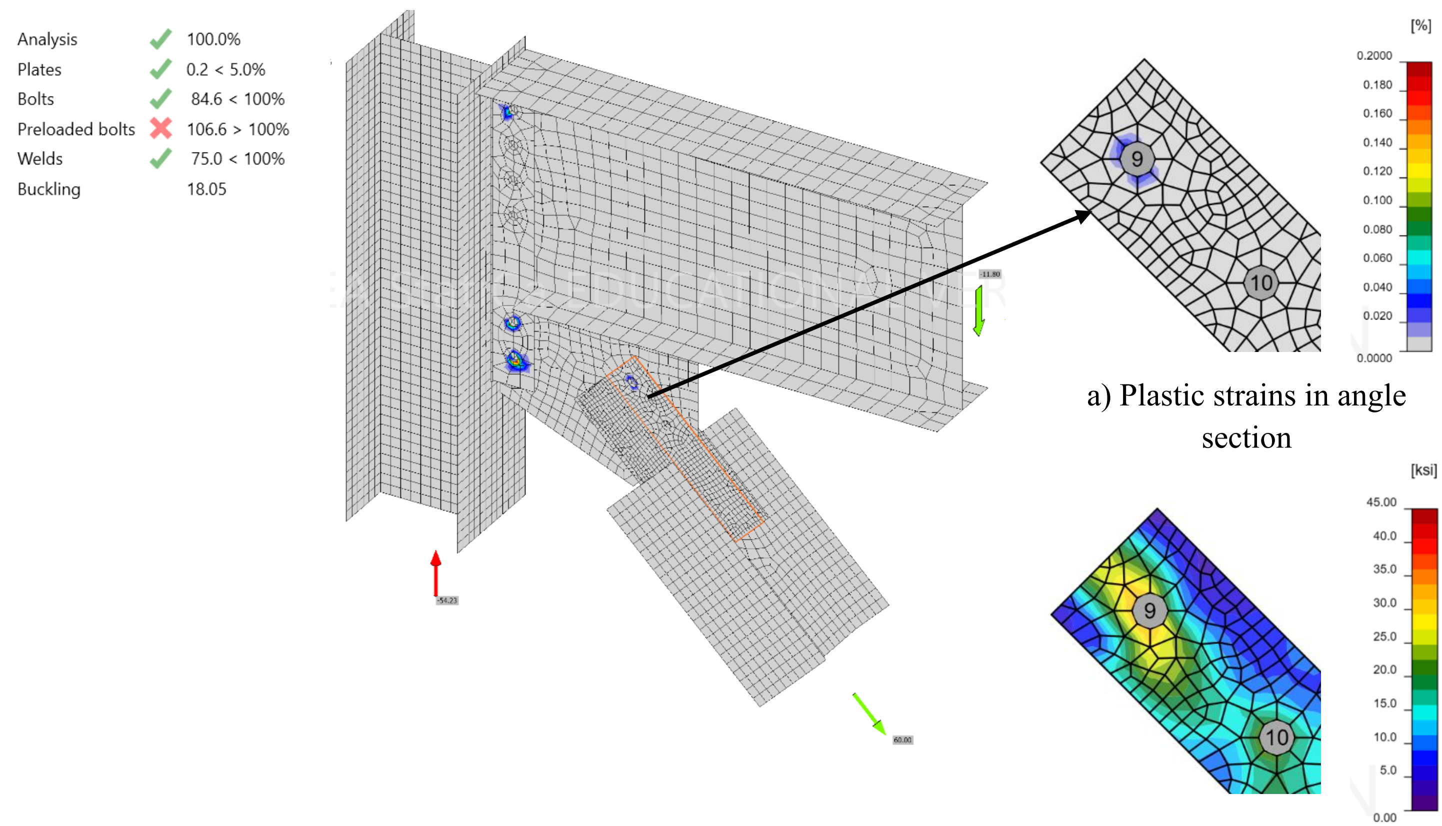
Figure 15 - von Mises stresses in connection for the action of design axial compression load in brace - CBFEM

6. Evaluation of limit states in CBFEM for tension load in brace

The calculations according to AISC Specifications are performed in accordance with the provisions for load and resistance factor design (LRFD) to obtain the results for several limit states of the connection. In the case of CBFEM, the limit states will be investigated individually with several iterations, by observing the plastic strains and equivalent stresses, and then the capacities will be reported accordingly from IDEA StatiCa Version 23.0.

The maximum permitted loads were determined iteratively by adjusting the applied load input to a value that the program deems safe but if increased by a small amount (1 kips) until the results would deem unsafe. As mentioned earlier, the focus of this study was to evaluate the limit states related to connection only.

The first limit state investigated is bolt bearing and tear out at angle section, brace web and on gusset plates. This limit state is found to be the governing limit state according to both AISC and CBFEM. The bolt bearing/tearout capacity according to CBFEM is 60 kips. The bolt bearing/ tearout capacity according to AISC 360 is 64 kips. The difference between the two capacities is 6.4% and is conservative according to CBFEM. The plastic strains and von Mises stresses in connection for evaluation of bolt bearing and tear out at angle section for 60kips axial tension load is shown in Figure 16.



a) Plastic strains in angle section

b) von Mises stresses in angle section

Figure 16 - Plastic strains and von Mises stress in connection for evaluation of bolt bearing and tear out at angle section - CBFEM

For the case of bolt bearing check in CBFEM, it is considered for each bolt individually, and not for the whole connection, which results in safer and more conservative results than AISC in general.

In CBFEM, failure in the members and plates due to yielding and rupture limit states are measured based on the 5% plastic strain limit. The plastic strain starts at the bolt holes and the stresses are based on von Mises stresses which is a combination of normal stresses and shear stresses. For **tensile rupture of angle sections** connecting the brace web to gusset plate, the value as per CBFEM (110 kips*f*) is very conservative when compared to value as per AISC (240 kip). The plastic strain is as shown in Figure 17.

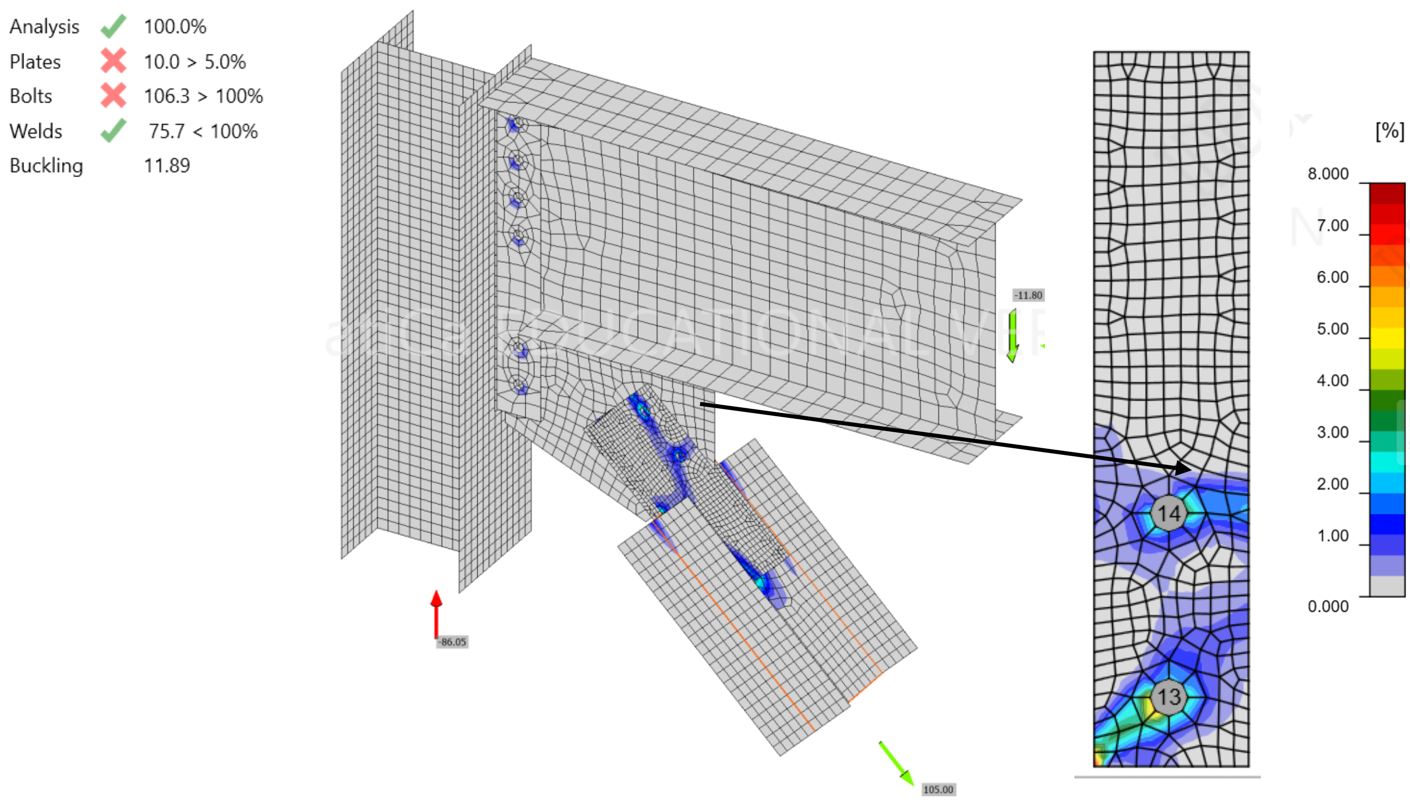


Figure 17 - Plastic strains in connection for evaluation of tensile rupture of angles - CBFEM

This difference is mainly due to criteria used in the CBFEM analysis, the 5% plastic strain. In our analysis, failure or capacity is reached once the plastic strain at any point is exceeded. The load can be increased further where more plastic strains may be obtained around the hole, but with not much load increase due to the bilinear stress-strain curve used in the analysis.

In CBFEM, it can be observed that the **block shear limit state** at certain load exists in some members and not in others. For the block shear rupture in brace web, value as per CBFEM (110 kip) as shown in Figure 18 is a conservative value when compared to the value as per AISC (124 kips), with a 12.72% difference.

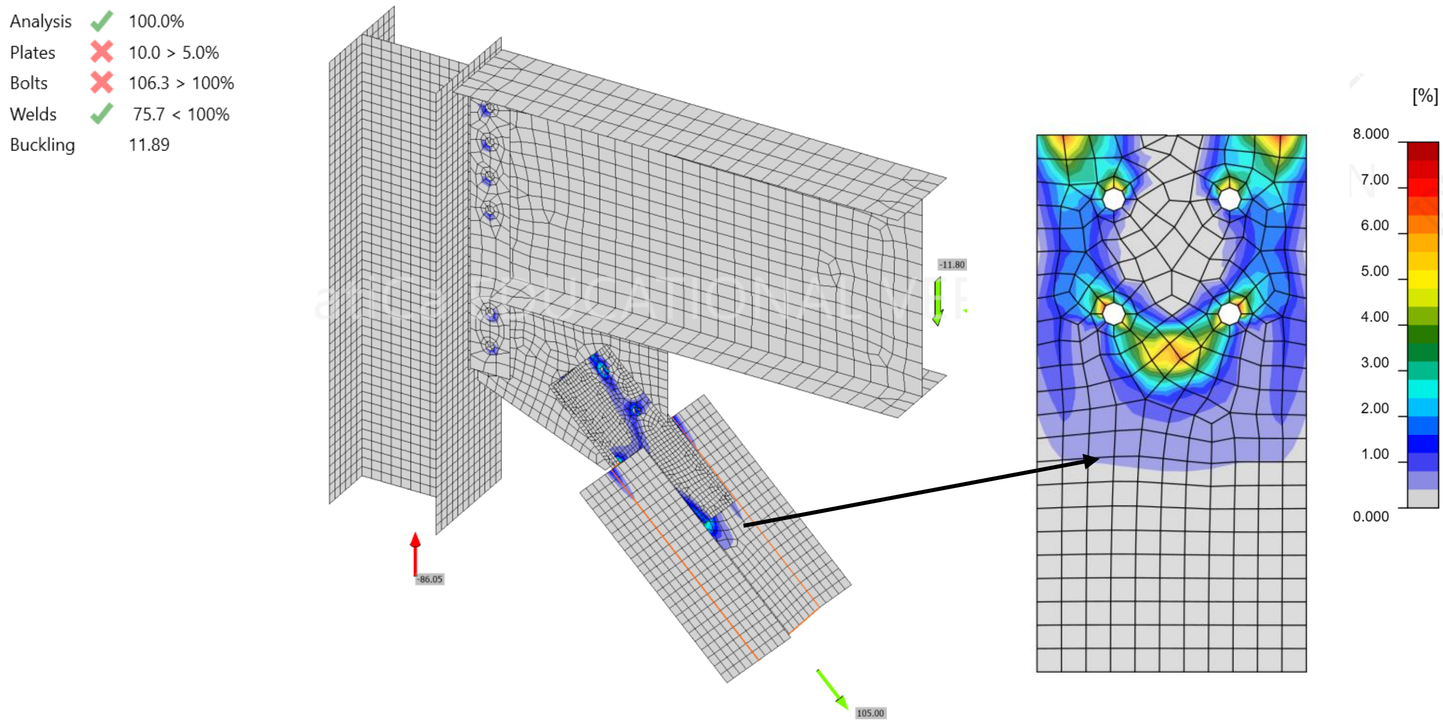
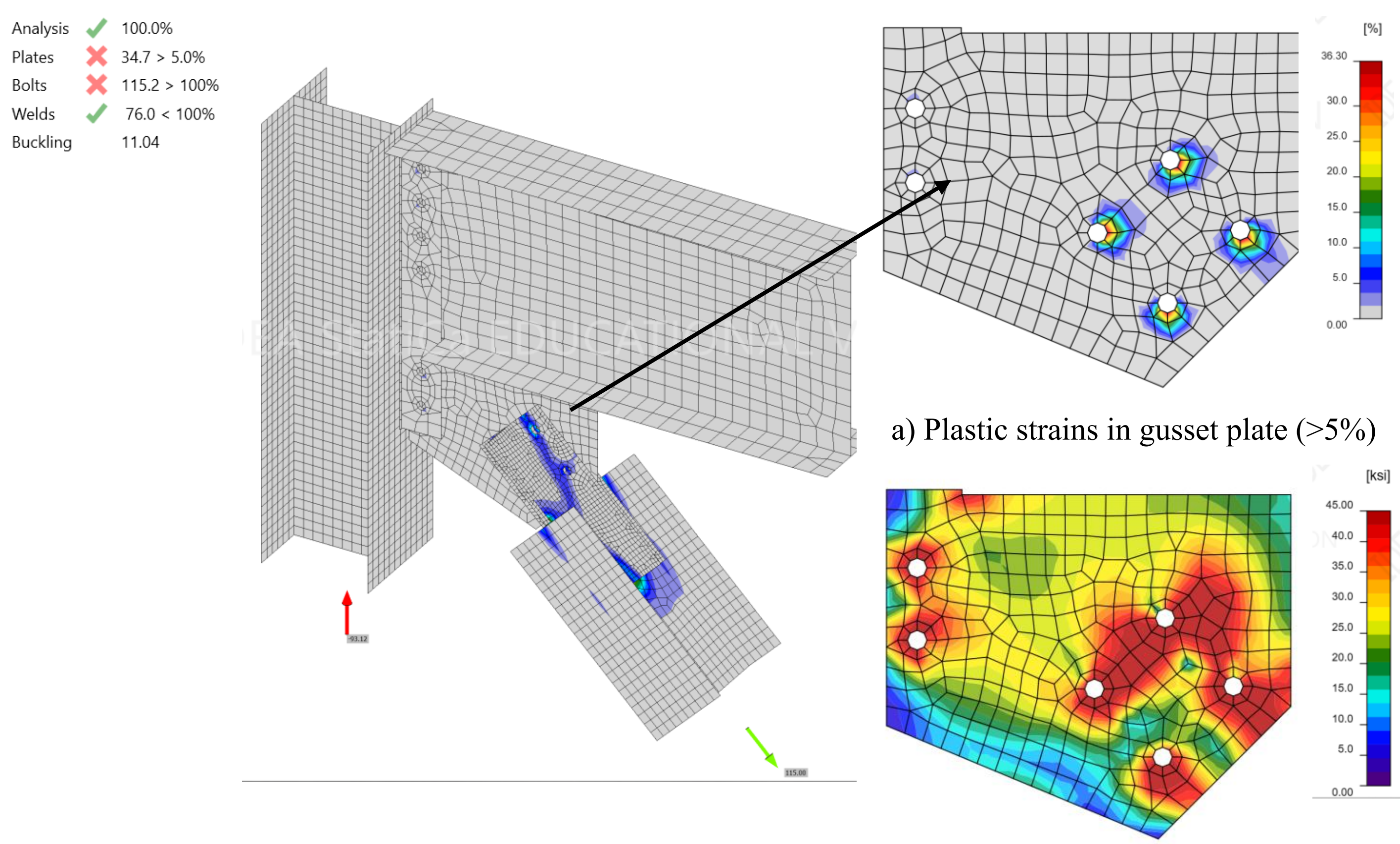


Figure 18 - Plastic strains in connection for evaluation of block shear in brace web - CBFEM

Finally, in the case of block shear rupture in gusset plate, value as per CBFEM (115 kip) as shown in Figure 19 is in quite close agreement to the value as per AISC (164 kip).

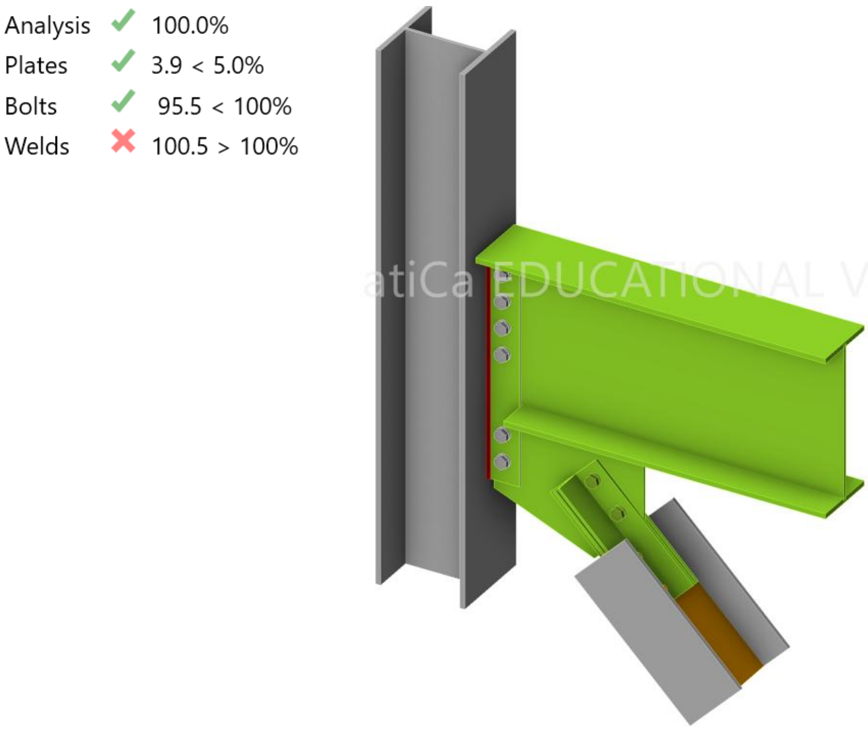


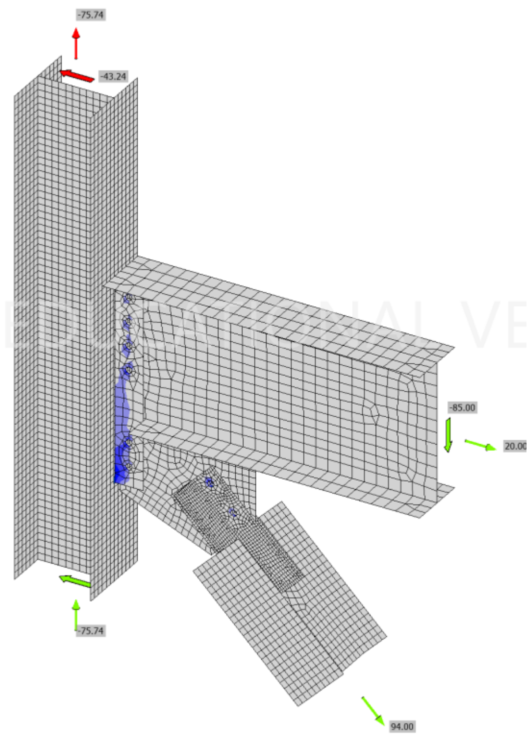
a) Plastic strains in gusset plate (>5%)

b) von Mises stress in gusset plate

Figure 19 - Plastic Strains in connection for evaluating block shear in gusset plate – CBFEM

The evaluation of ¼” fillet weld connecting the fin plate to the column flange is performed as shown in Figure 20. The brace is loaded such that the weld utilization percentage at the required weld is at least 100% in CBFEM. To evaluate the fillet weld connecting the gusset to beam, the weld at fin plate to column was made CJP weld in CBFEM, so the evaluation of the considered weld can be performed effectively as shown in Figure 21. Overall, the ¼” fillet weld size in CBFEM agrees with that as per AISC as given in Table 2.





a) Overall Check - CBFEM

b) Plastic strains - CBFEM

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 6.88 kip < $F_n =$ 6.91 kip

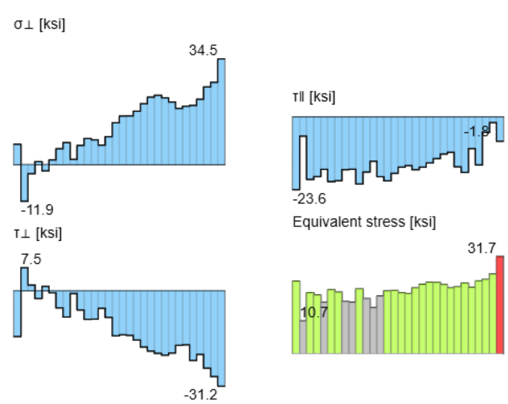
Where:

$F_{nw} =$ 62.6 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 80.4° – angle of loading measured from the weld longitudinal axis

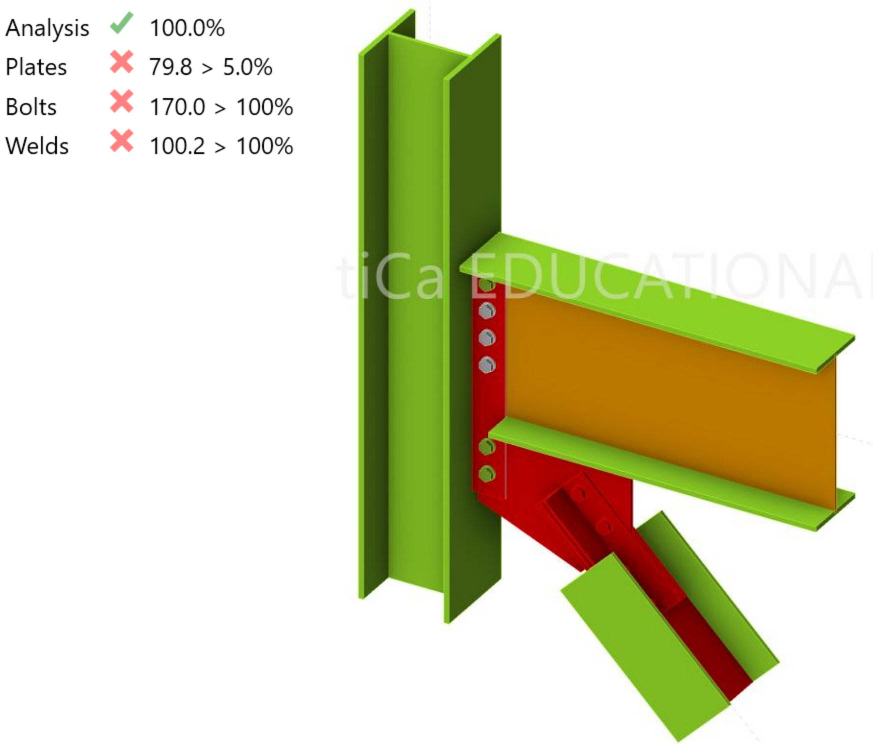
$A_{we} =$ 0.1465 in^2 – effective area of weld critical element

$\phi =$ 0.75 – resistance factor for welded connections

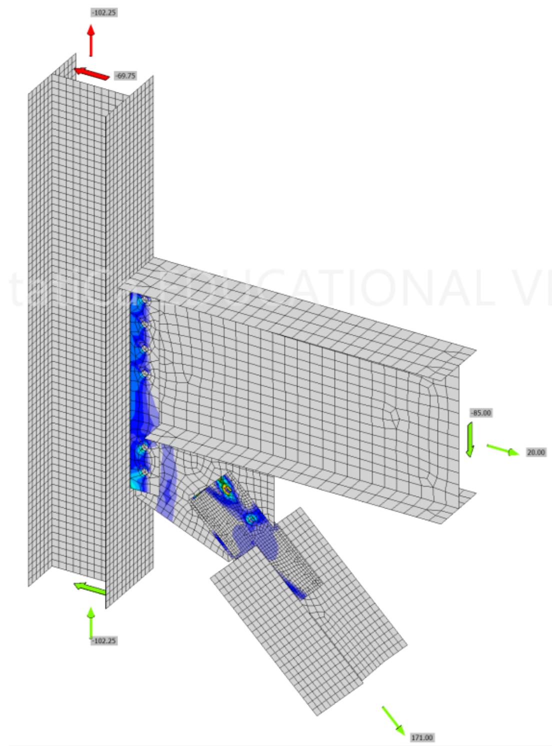


c) Stresses in considered weld - CBFEM

Figure 20 – Evaluation of fillet weld connecting fin plate to column – CBFEM



a) Overall Check - CBFEM



b) Plastic strains - CBFEM

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.92 kip < $F_n =$ 4.93 kip

Where:

$F_{nw} = 43.7$ ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 10.8° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.1502$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

c) Stresses in considered weld - CBFEM

Figure 21 – Evaluation of fillet weld connecting gusset plate to beam – CBFEM

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16^{ths} weld size (in)	
			AISC	CBFEM
1	94	Fin plate to column	4	4
2	171	Gusset plate to beam	4	4

where,
P_{brace} is axial load in brace for which weld percentage utilization is 100% in CBFEM.

Table 2 – Comparison of fillet weld size as per AISC and CBFEM

7. Evaluation of limit states in CBFEM for compression load in brace

Due to cyclic loading of an earthquake, a brace can buckle in compression which results in significant loss of the brace strength as well as of the connecting gusset plate. **Buckling of gusset plate** required by AISC can be checked by a buckling multiplier factor obtained using CBFEM. Currently, it is the only measure, and it is hard to differentiate between the buckling resistance of various connecting parts, e.g., buckling of gusset plate on the Whitmore section or gusset plate sidesway buckling.

The **check of buckling for connection is performed for the design compression load** of 53.2kips in the brace, such that load on beam and column are balanced by “loads in equilibrium” in CBFEM as shown in Figure 22. Since the gusset plate is connected to two sides in the presented connection – welded to bottom beam flange and bolted to fin plate, the buckling can be classified as “**local buckling**”. The buckling factor for the critical mode is 7.94 which is more than the prescribed buckling factor of 4 for a gusset plate with grade of A572-50 subjected to local buckling. If the buckling factor for critical model was less than the prescribed buckling factor, then the next step would have been to increase the thickness of the gusset plate.

Hence, it can be concluded that buckling was not an observed limit state in both AISC and CBFEM.

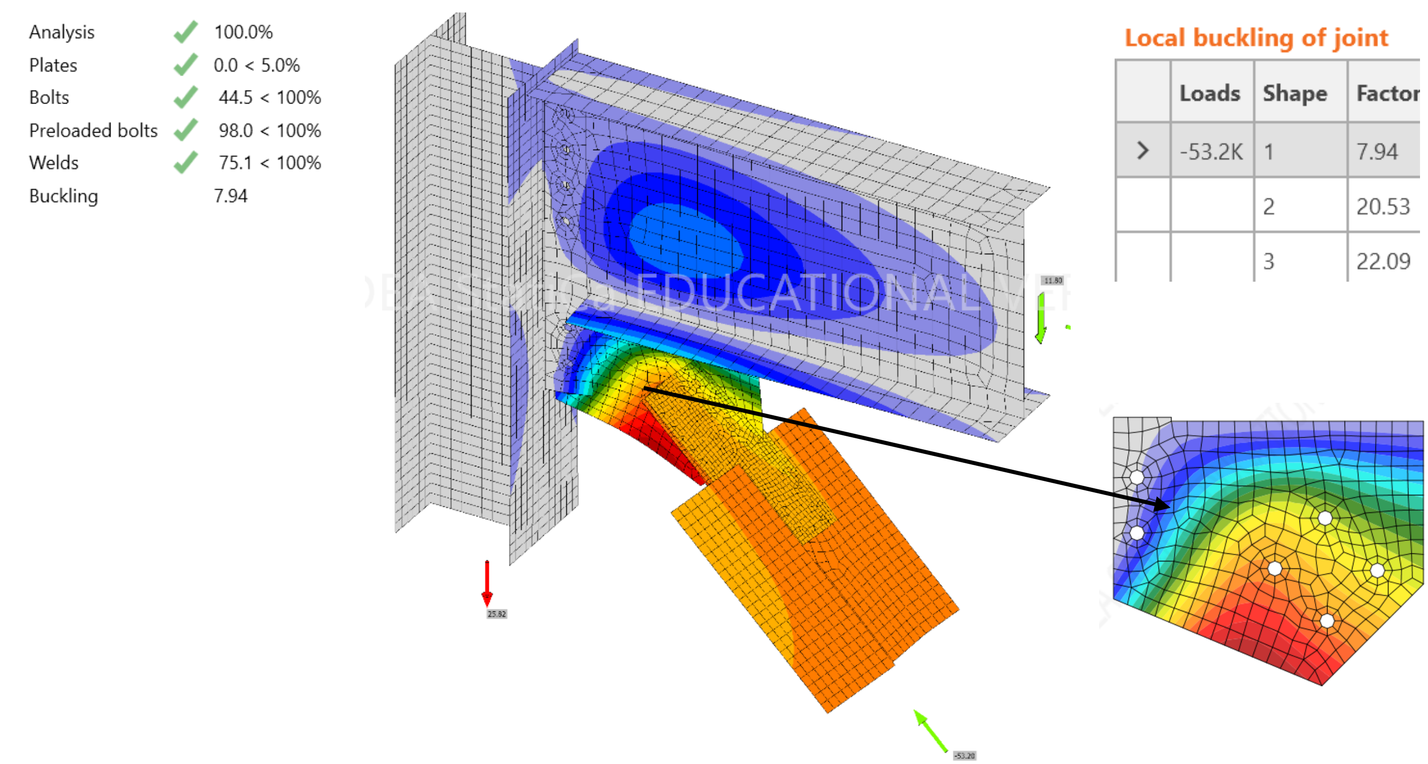


Figure 22 - Buckling analysis of wide flange brace corner connection - CBFEM

8. Variation Studies

To evaluate the buckling of the connection in detail, a variation study for four different gusset plate thicknesses was performed for the design compression load of 53.2kips in the brace member in CBFEM. The results are tabulated in Table 3 and the plot of results is shown in Figure 23 comparing the buckling capacities obtained from AISC with CBFEM.

It can be concluded that the CBFEM provides conservative results for buckling capacity in IDEA StatiCa connection which is based on linear buckling analysis.

Sr No.	t_{gp} (in)	Brace Size	Results from AISC		Results from CBFEM			
			P_{b_AISC} (kips)	P_{comp} (kips)	α_{cr}	P_{b_CBFEM} (kips)	Buckling observed in	Check for buckling
1	3/8	W10x33	134	53.2	7.9	105	Gusset plate	OK
2	1/2	W10x33	188	53.2	10	133	Gusset plate	OK
3	¾	W10x33	292	53.2	14.5	193	Brace member	OK
4	1	W10x33	395	53.2	17.3	230	Brace member	OK

where,

- t_{gp} is gusset plate thickness.
- P_{b_AISC} is buckling capacity as per AISC.
- P_{comp} is the design compression load in brace as per AISC (independent of gusset plate thickness).
- α_{cr} is the critical buckling factor (buckling factor for mode 1) as per non-linear buckling analysis in IDEA StatiCa connection.
- P_{b_CBFEM} is buckling capacity as per CBFEM, calculated as $P_{b\_CBFEM} = (P_{comp} x\ \alpha_{cr})/4$.

  % Δ_{AISC_CBFEM} is the percentage difference in the buckling capacity of AISC and CBFEM.

Table 3 – Parametric Study for buckling of corner connection in OCBF

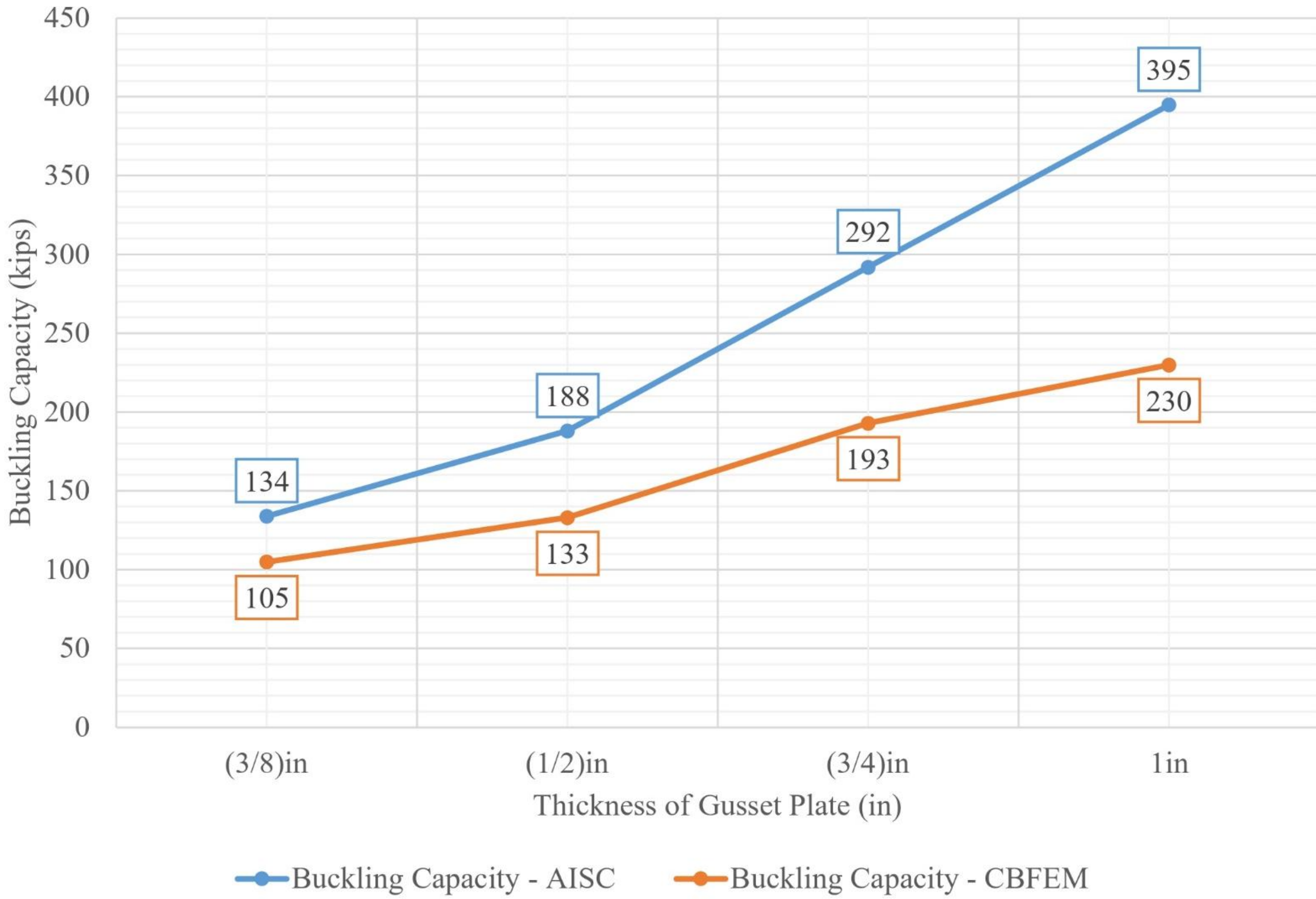


Figure 23 – Curve of Buckling capacity vs thickness of gusset plate

To evaluate the impact on the CBFEM results on the percentage utilization of the connections due to variation of maximum element size in mesh, mesh sensitivity analysis was performed. Eight specific maximum element sizes: 0.35 in., 0.75 in., 1in., 1.25 in., 1.5 in., 1.75 in. and 2 in. are considered and the utilization of bolts for the action of axial tension load of 105 kips in brace. As per "default" setting for model and mesh in IDEA StatiCa, the minimal element size was kept as 0.3 in (default value), while the maximum element size was varied for the six specific element sizes as mentioned above. The results obtained are plotted as shown in Figure 24. Based on the observations, the values for the bolt utilization (%) – bearing type bolts and slip critical bolts are approximately constant. Smaller maximum element sizes tend to make the analysis time longer.

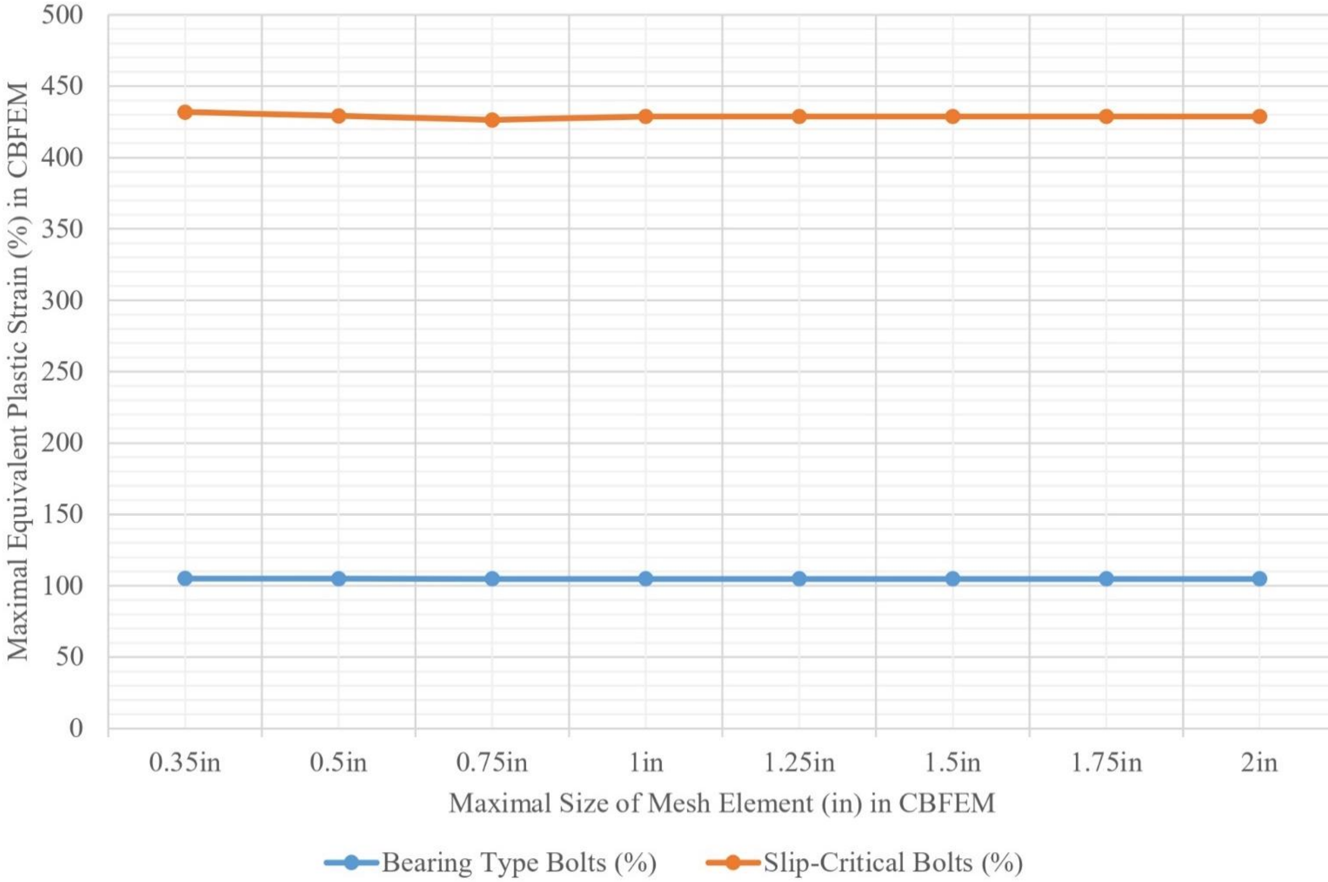


Figure 24: Mesh Sensitivity Analysis for corner connection in OCBF

9. Comparison of limit state values from CBFEM with AISC (Only for the purpose of internal review)

Sr No.	Limit State of	AISC (kips)	CBFEM (kips)	% difference	Related to	Comment
1	Bolt bearing and tear out on angles	64	60	-6.3%	Connection	CBFEM is slightly conservative.
2	Bolt bearing and tear out on brace web (SS)	64	60	-6.3%		CBFEM is slightly conservative.
3	Bolt bearing and tear out on plate (SS)	64.5	60	-6.3%		CBFEM slightly conservative by 6.3%.
4	Compressive strength of brace	72	NA	NA	Member	
6	Block shear rupture of bottom brace web	124	110	-19%	Connection	CBFEM very conservative.
7	Beam web local crippling	131	NA	NA	Member	
8	Block shear rupture in the gusset plate	164	115	-42%	Connection	CBFEM very conservative.
9	Block shear rupture in the angles	170	115	-49%		CBFEM very conservative.
10	Tensile rupture in the angles	240	110	-129%		CBFEM very conservative.
10	Beam web local yielding	265	NA	NA	Member	
11	Tension yielding in the angles	272	110	-147%	Connection	CBFEM very conservative.
12	Tensile Rupture strength of bottom brace web	285	110	-159%		CBFEM very conservative.
13	Column web local crippling	286	NA	NA	Member	
14	Column web local yielding	445	NA	NA	Member	

10. Conclusion (For presented connection only)

1. CBFEM can predict the actual behavior and failure modes for the presented connection.
2. Joint design resistance analysis in CBFEM offers insight into the reserve in the connection resistance which is based on plastic strains and von-Mises stresses. For the presented connection, design resistance for the action of compression loads in brace was observed to be 23% higher than that for the case of axial tension load in brace as per per CBFEM, which indicates that the limit states of compression are not governing.
3. Traffic light type format visualization for overall check in CBFEM offers easy visualization of failure in connection or member, after running the analysis for assigned loads in stress-strain analysis.
4. For the case of bolt bearing check in CBFEM, it is considered for each bolt individually, and not for the whole connection, which results in safer and more conservative results than AISC for this case.
5. The gusset plate limit states including yielding and tension rupture are based on the 5% plastic strain limit as per CBFEM.
6. For block shear limit state, it is observed in the gusset plate and along the connecting brace section. Also, the block shear computation in CBFEM is based only on yield strength of steel, while the equation in AISC is based on both yield strength of steel and ultimate strength of steel. But, since the resistance factor for block shear is 0.75 according to AISC 360, which is used for both yielding and rupture components; and 0.9 resistance factor is used for yielding, they are usually balanced.
7. The buckling limit state of the gusset plate is evaluated for the action of design compression load in the brace. It was not observed as a limit state in AISC and CBFEM.
8. The limit states for beam such as beam web buckling, web crippling and shear yielding occurs at the higher loads, therefore they are not checked in the CBFEM, since the model will not converge at such higher loads and all the limit states would occur before this limit state.
9. Some mesh dependency was observed for the case of percentage utilization of the connections (bolts and welds) due to variation in the maximum element size for the mesh.

11. References

1. AISC 360. (2016). *Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago, Illinois.*
2. AISC 341. (2016). *Seismic Design Manual. American Institute of Steel Construction, Chicago, Illinois.*
3. IDEA StatiCa. (n.d.). *IDEA StatiCa Support center- FAQ*. https://www.ideastatica.com/support-center-faq