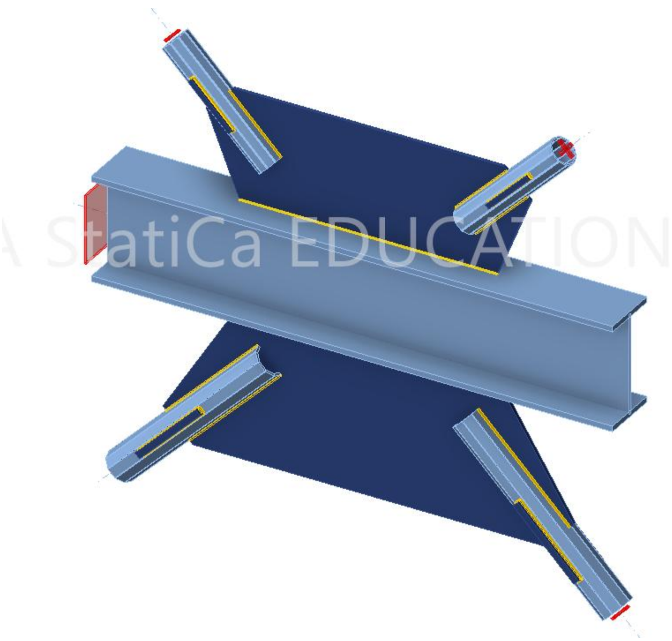
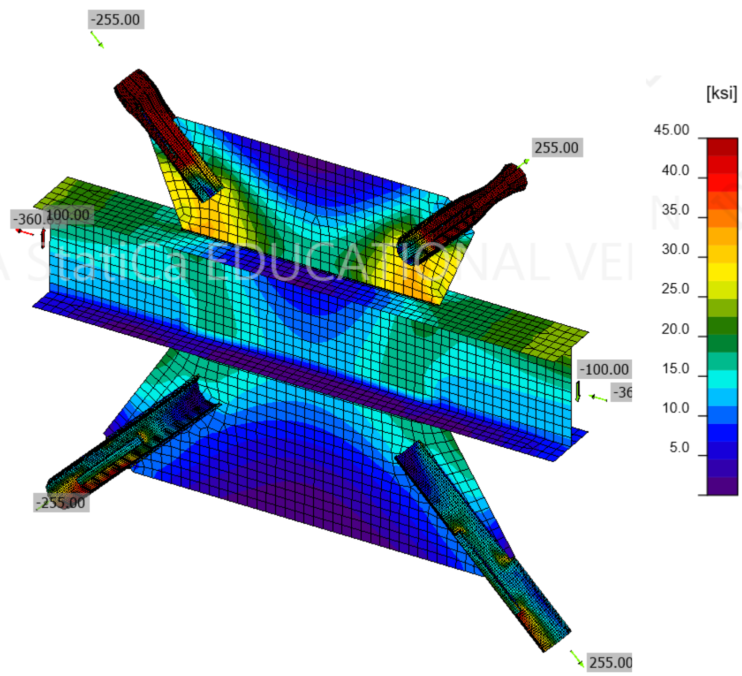
Connection in a Two-Story X-Brace Configuration of Ordinary Concentrically Braced Frame (OCBF) System





This is the **second verification example** in CBFEM from a **series of seismic vertical brace connections**. It compares a **connection in multistorey X-brace configuration** of 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. Problem Description

For steel structures with more than two stories and having an X-brace configuration, the commonly used connection between a wide flange beam and braces is as shown in Figure 1, which consists of a gusset plate at the top and bottom flanges of the beam, fillet welds between the brace members and the gusset plates, reinforcing plates on brace members, and erection bolts in the braces.

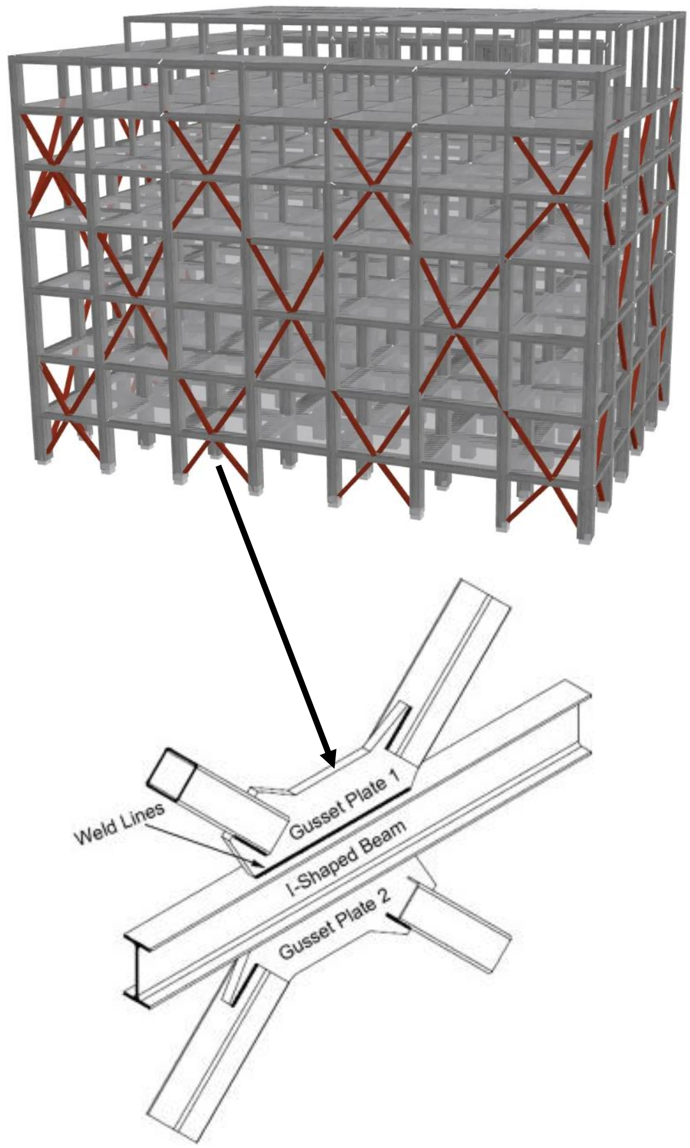


Figure 1 - Commonly used wide flange beam to HSS brace connection in two-story configurations (Roostaeyan et al., 2020) (Seismosoft, 2022)

Vertical braces are diagonal members installed in the vertical direction of the steel frame. The function of the braces is to transfer the applied lateral forces between the different floors down to the foundations. They are usually provided between two columns from the ground-up, just like a vertical cantilever truss. For the vertical bracing to be effective, they must cross each floor throughout the entire height of the steel frame.

The objective of this example is **to verify the Component-Based Finite Element Method (CBFEM)** for a connection in a two-story X-braced configuration of **an Ordinary Concentric Braced Frame (OCBF)**. The connection details investigated is shown in Figure 2. The results obtained from calculation method which are based on (AISC 360, 2016) Specifications and AISC 341-16 provisions are compared with results obtained from the CBFEM analysis **using stress-strain analysis** in **IDEA StatiCa software version 23.0.**

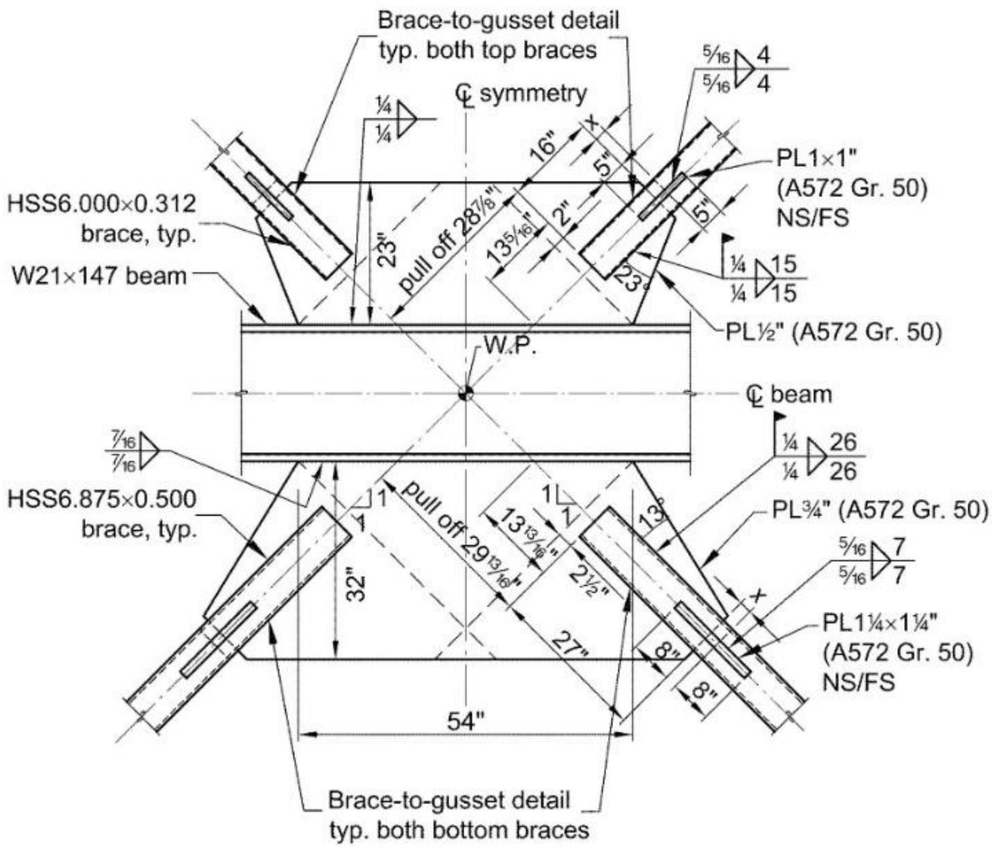


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Figure 2 - Details for connection in two-story X-brace configuration in OCBF (AISC 341, 2016)

The details for member and connection are as given below,

Member Details

1. **Beam cross-section**

- W21x147
- ASTM A992

2. **Brace cross-section**

- Top brace as HSS6x0.312
- Bottom brace as HSS6.875x0.5
- ASTM A500, Grade C Round

Connection Details

1. **Welds**

- E70xx electrode
- 1/4” double sided fillet weld between top gusset plate to top flange of beam.
- 7/16” double sided fillet weld between bottom gusset plate and bottom flange of beam.
- 1/4” rear sided fillet weld between top brace and top gusset plate.

Plate Details

1. Gusset Plate

- Top gusset plate as ½”
- Bottom gusset plate as ¾”
- ASTM A572-50

2. Reinforcing Plate on HSS Brace

- PL 1”x1” on top brace
- PL 1-1/4”x1-1/4” on bottom brace
- ASTM A572-50
- 5/16” rear sided fillet weld between bottom gusset plate and bottom gusset plate.
- 5/16” double sided fillet welds between all reinforcing plates and HSS brace wall.

2. Modelling and Analysis of steel connection in IDEA StatiCa

The wall of the HSS brace can only develop a certain amount of load per linear inch before it fails in shear. Therefore, the capacity of the weld connecting the brace to the gusset plate must not exceed the HSS wall shear capacity. The maximum weld size can be determined by setting the shear capacity equal to the weld strength and then solving for the weld size. The number of welds must also be determined; it is determined based on the geometry of the connection. For example, HSS braces are slotted through their center to frame the gusset plate, thus providing four locations for welds as shown in Figure 3. The length of weld required for each of the welds is determined based on the ultimate axial tension load in the brace.

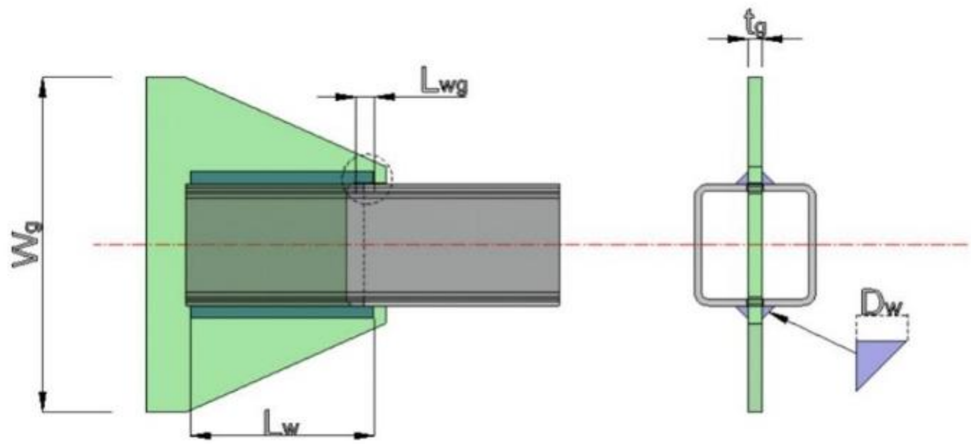


Figure 3 - Details of Slotted HSS brace connection (Afifi et al., 2020)

The **HSS braces** in IDEA StatiCa are modelled **as equivalent cold-formed section** with similar material properties, to ease the process of modelling, analysis and design of fillet welds between the stiffening plates and braces. The details for the HSS braces in the top gusset plate are as shown in Figures 4. To perform the CBFEM analysis of the given connection, it was modeled in the IDEA StatiCa Version 23 using **several operations in software**, shown in Figure 5.

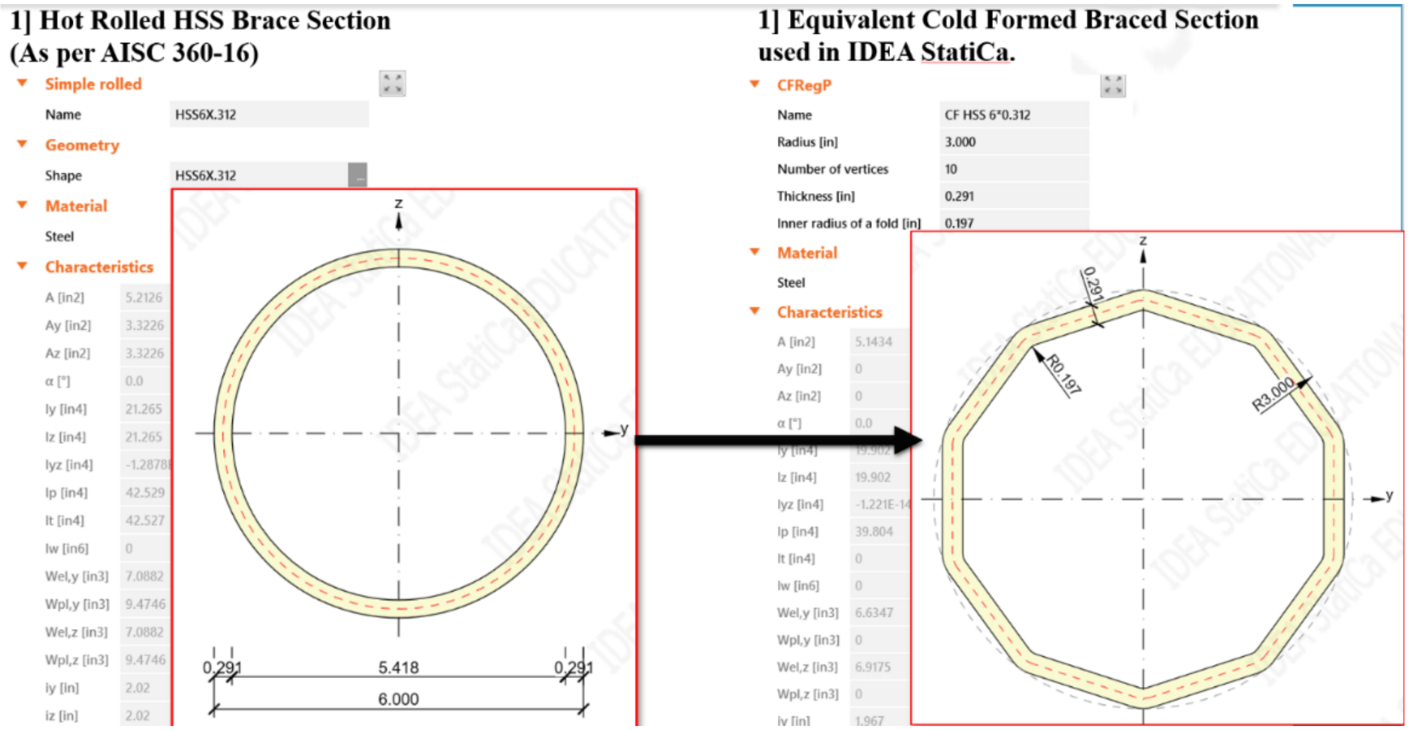


Figure 4 - The brace members as equivalent cold-formed sections in IDEA StatiCa

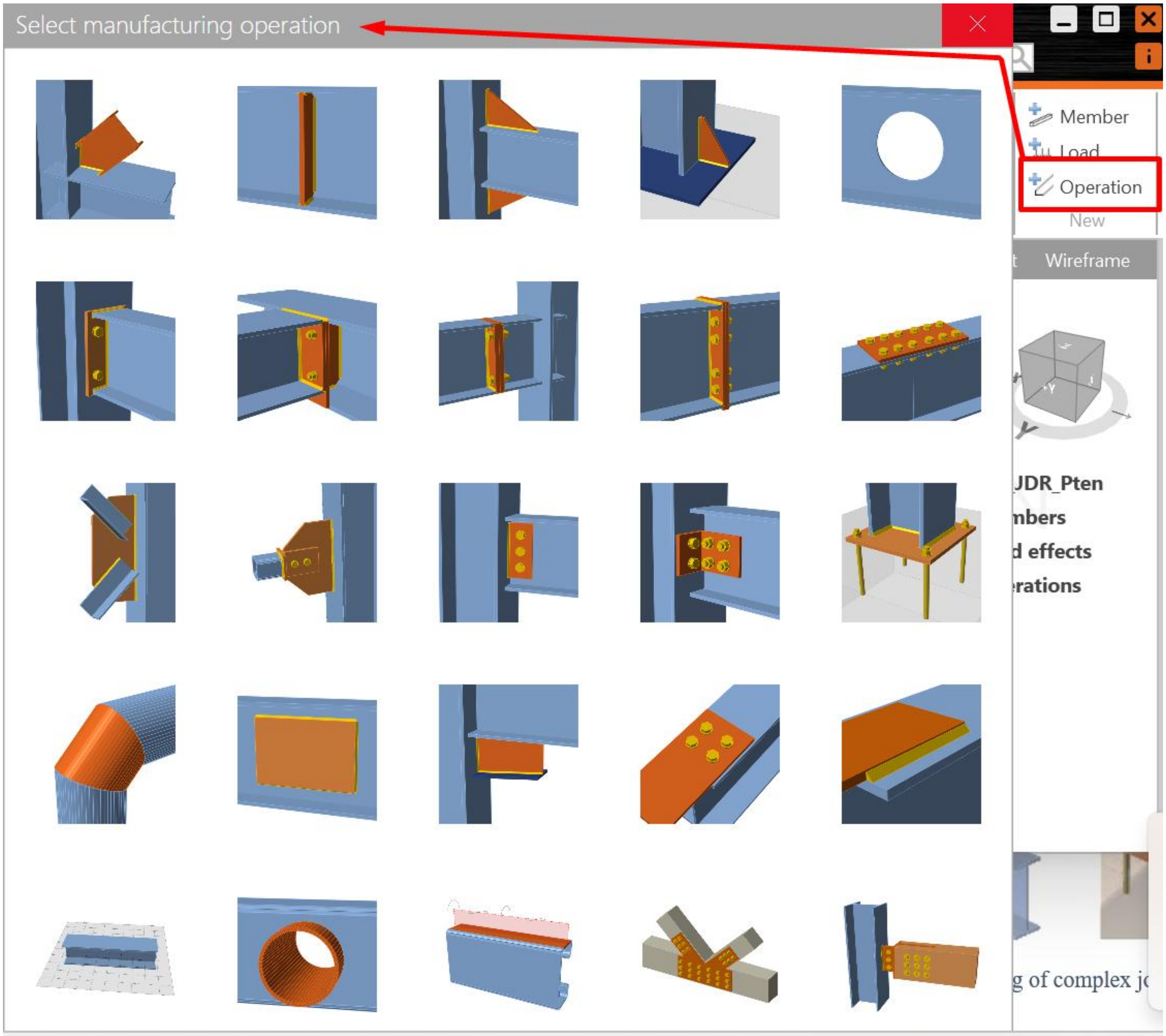
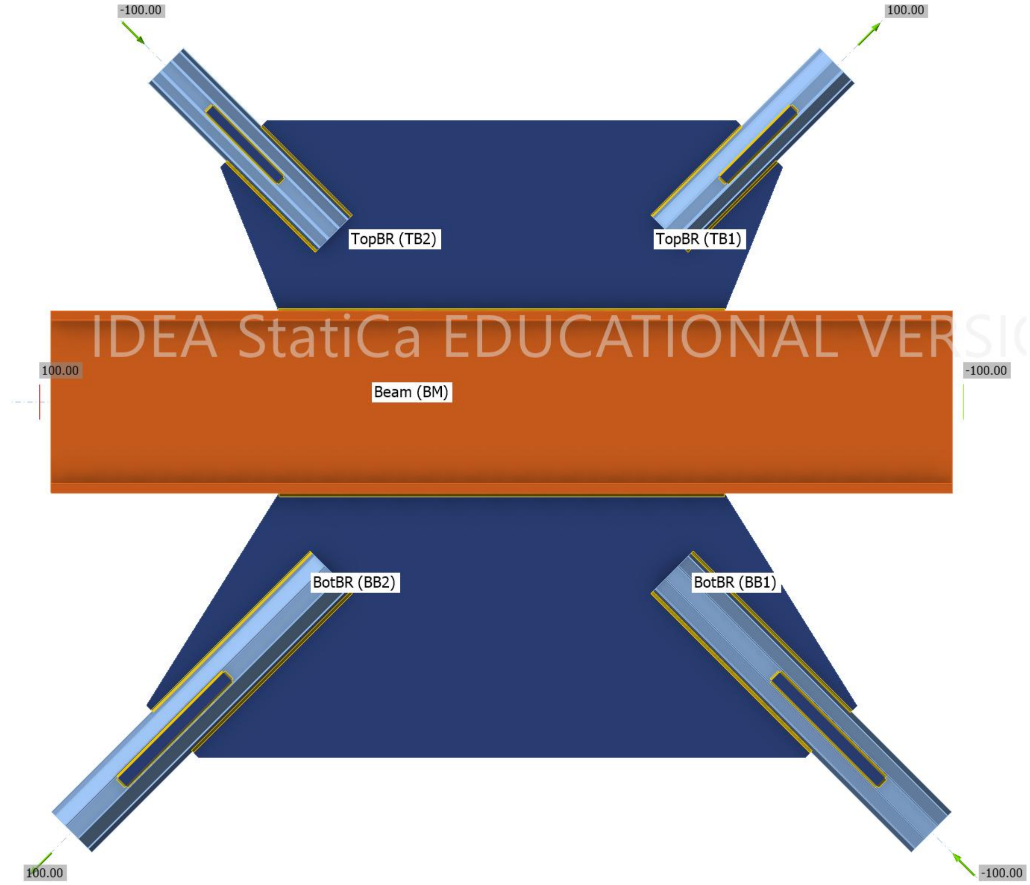
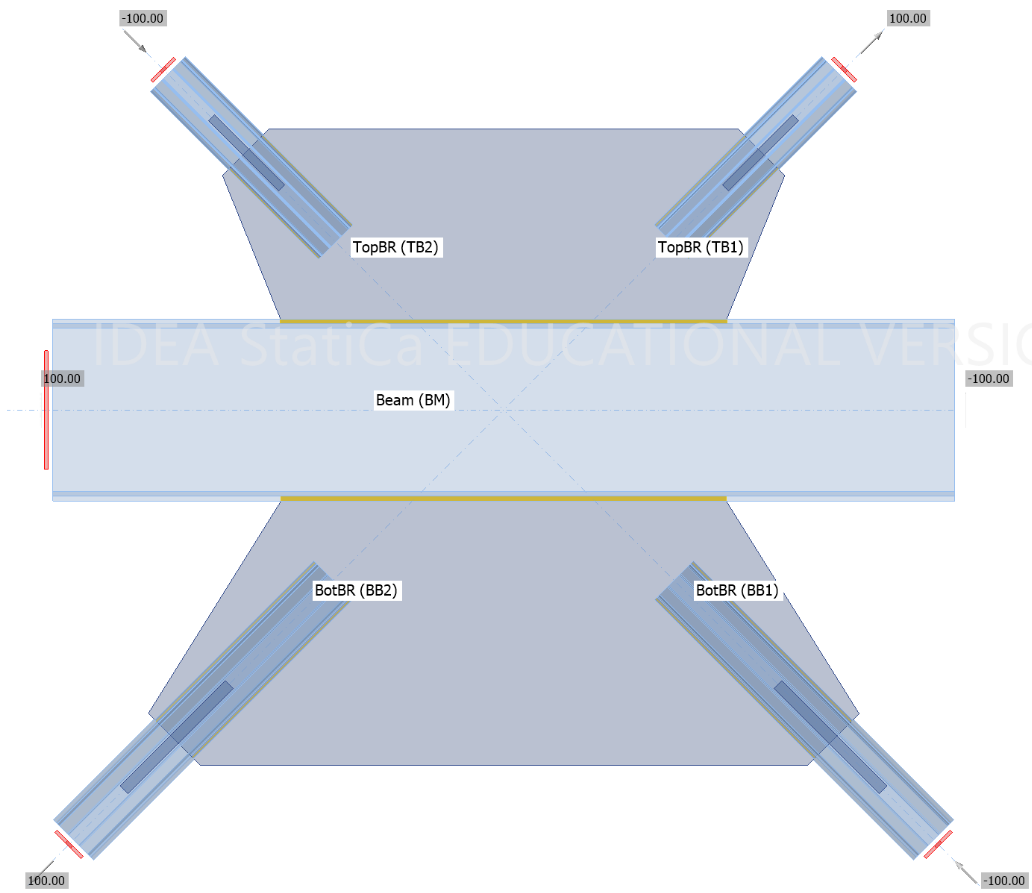


Figure 5 - Several manufacturing operations in IDEA StatiCa connection for modelling

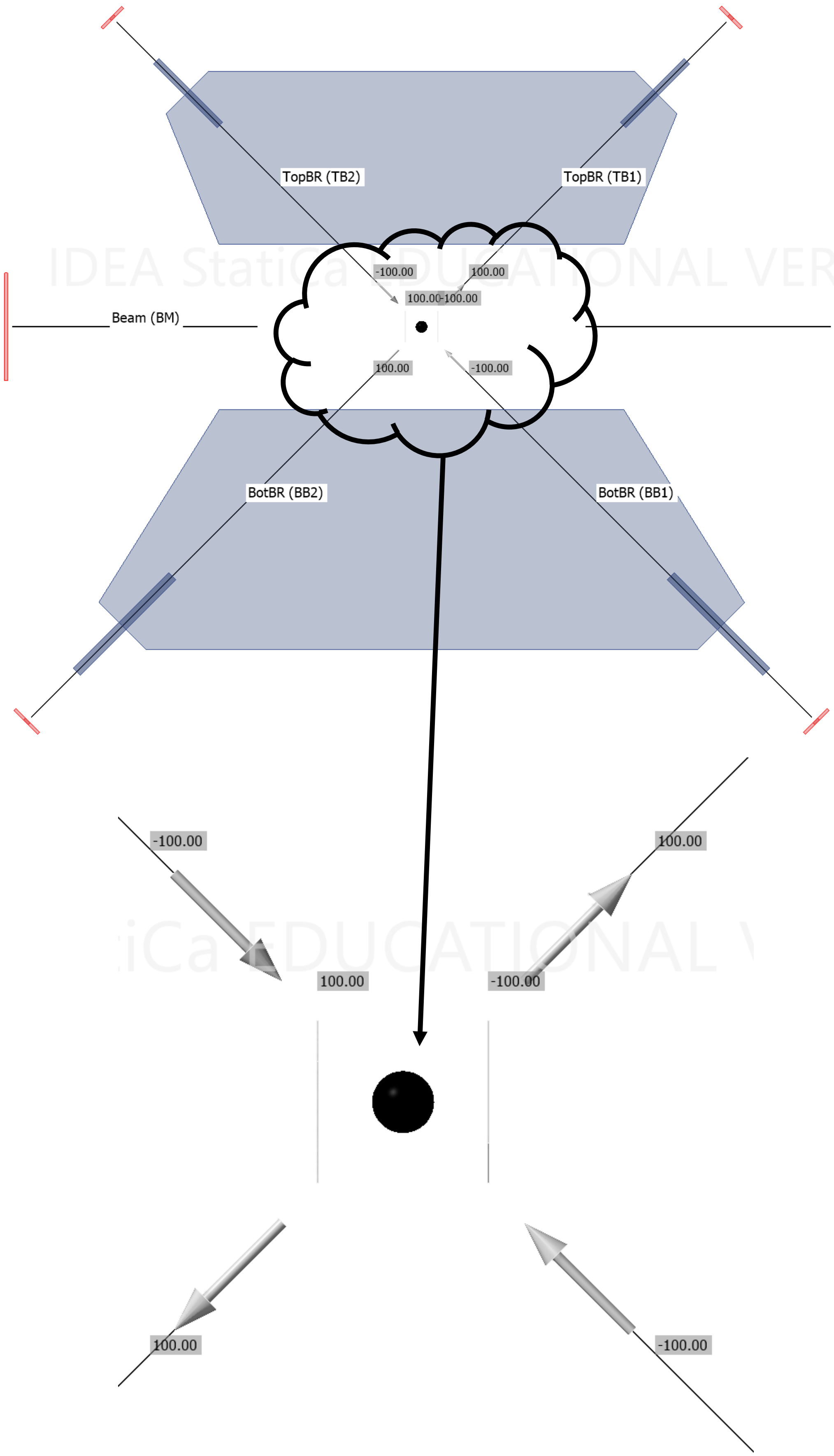
The solid, transparent and wireframe view of the base model are shown in Figure 6. The **beam** is selected as a **bearing member** and the loads on it are balanced based on the **"loads in equilibrium**" for **stress-strain analysis.** The model type of **N-V_y-V_z-M_x-M_y-M_z (Fixed)** is assigned to **beam**, while model type of **N-V_y-V_z (Pinned)** is provided to all the **four bracing members**, such that the forces are in the node, as seen in wireframe view in Figure 6.



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 6 - Configurations of the view of the connection for OCBF – Solid, Transparent and Wireframe view

The slotted holes in brace members can be easily modelled in the IDEA StatiCa by the combination of two operations – negative volume and cut of member at every brace member location as shown in Figure 7.

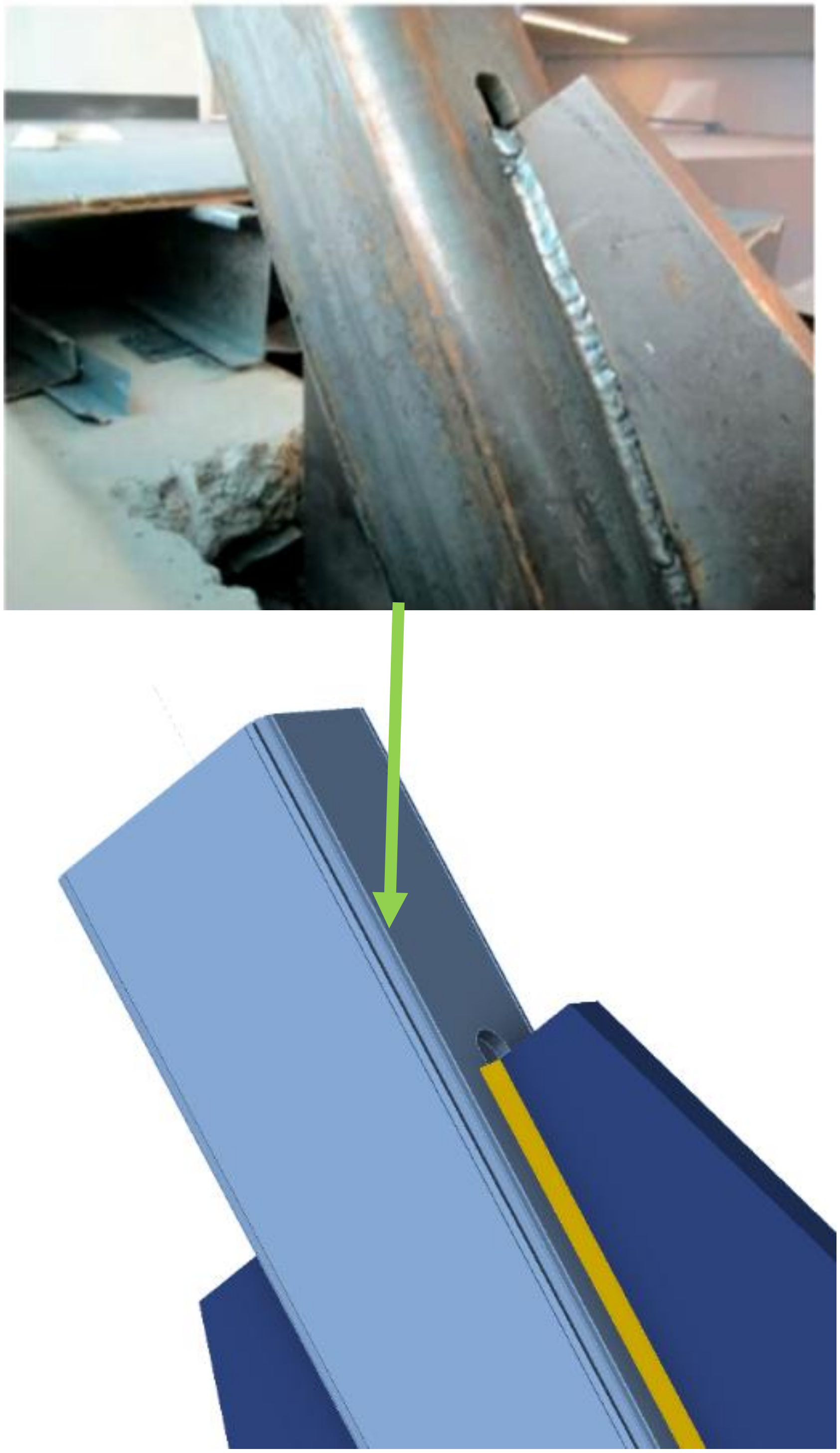


Figure 7 - Slotted Holes in HSS brace member to engage gusset plate (AISC 360, 2016)

3. Verification of Resistance in CBFEM – Joint Design Resistance (JDR)

Joint Design Resistance (JDR) analysis was performed to determine the axial load in brace connection for which the design resistance is 100%. The loads were gradually increased in all brace members starting from 100kips (with increments of 10kips each till 240kips) 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.

Based on the load resistance factor obtained from joint design resistance in CBFEM, a plot of load resistance factor vs applied axial brace load was prepared as shown in Figure 8. The load for 100% resistance factor was determined, which was close to 218kips in all the brace members (tension or compression), such that the brace members started to yield and the plastic strain in plates is more than 5% limit as set by CBFEM.

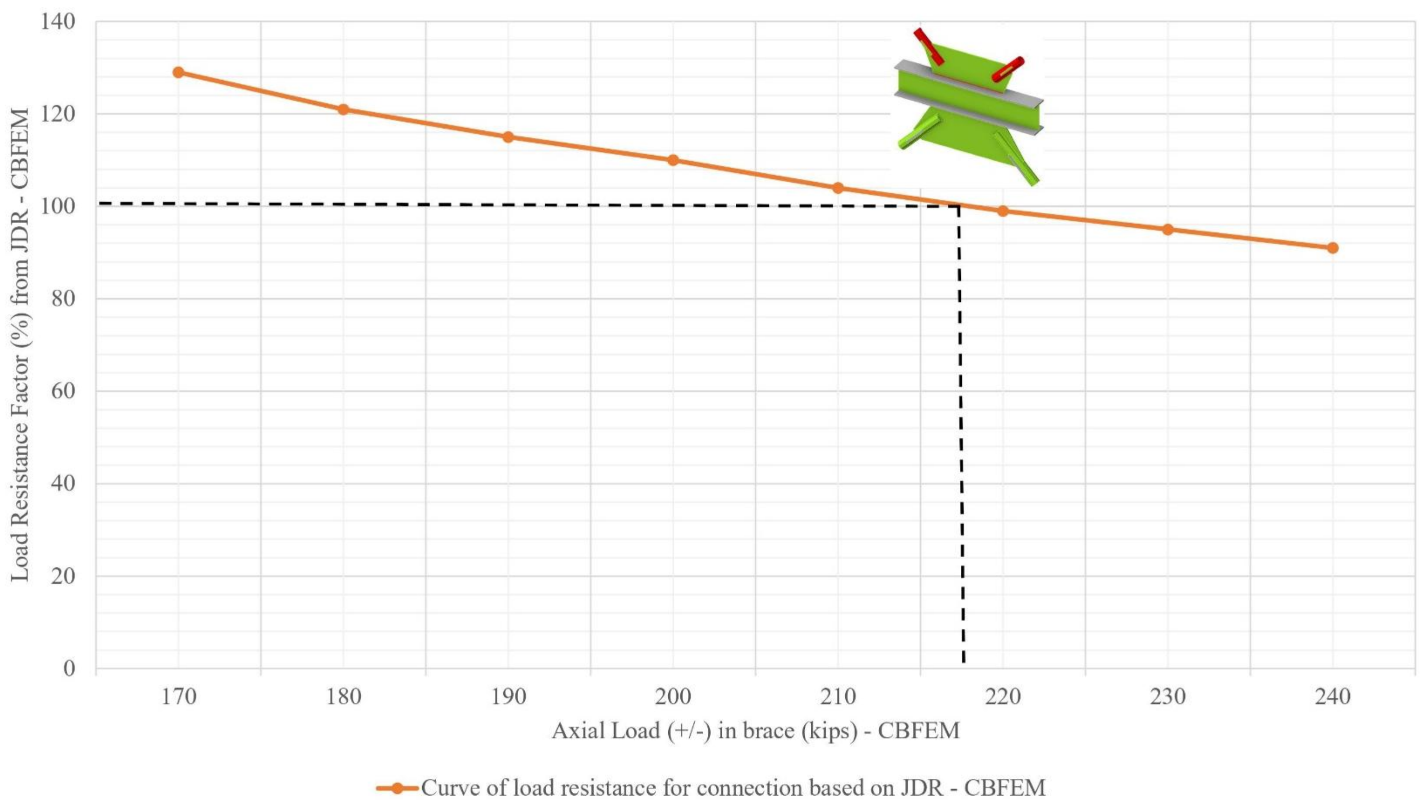


Figure 8 - Load Resistance for connection based on JDR from CBFEM

4. 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. The given connection is found to be safe for the action of design loads in CBFEM and the von-Mises stresses are as shown in Figure 9.

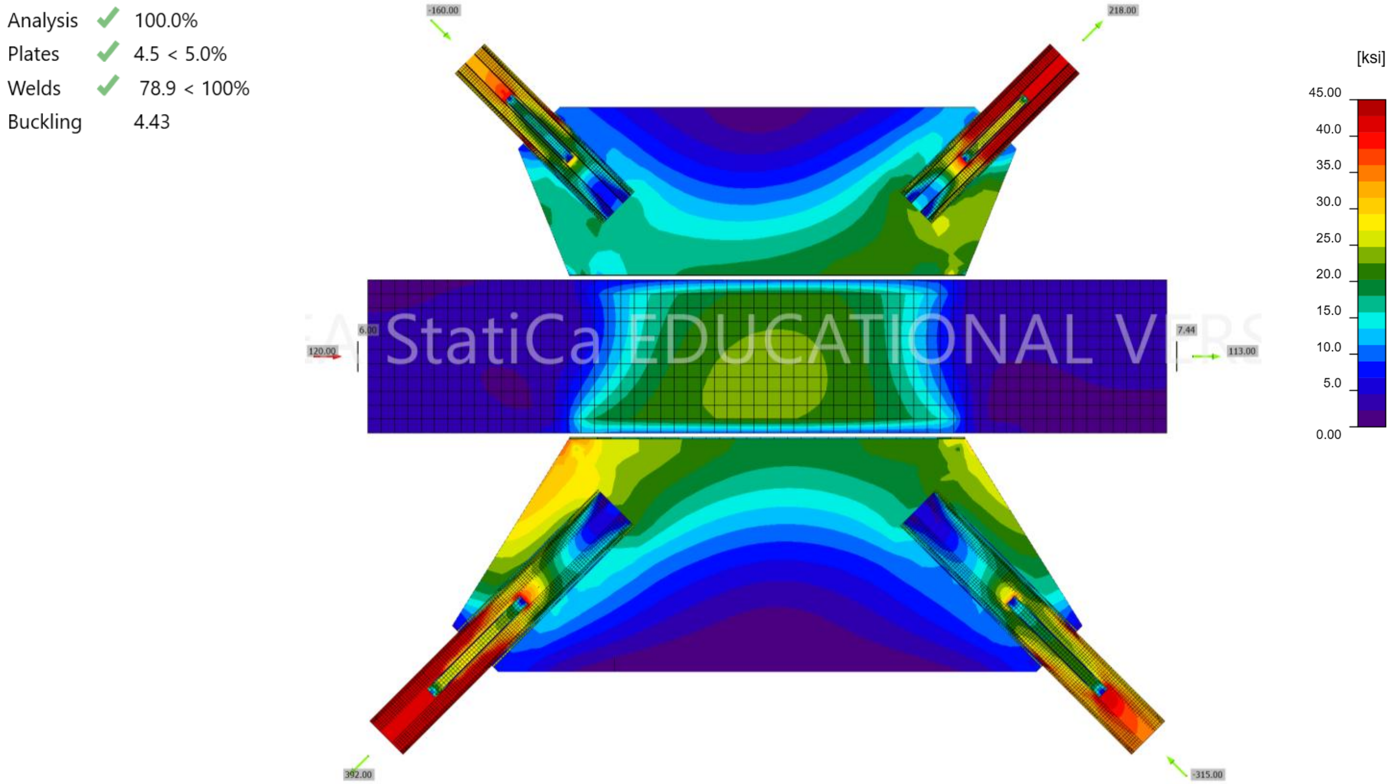


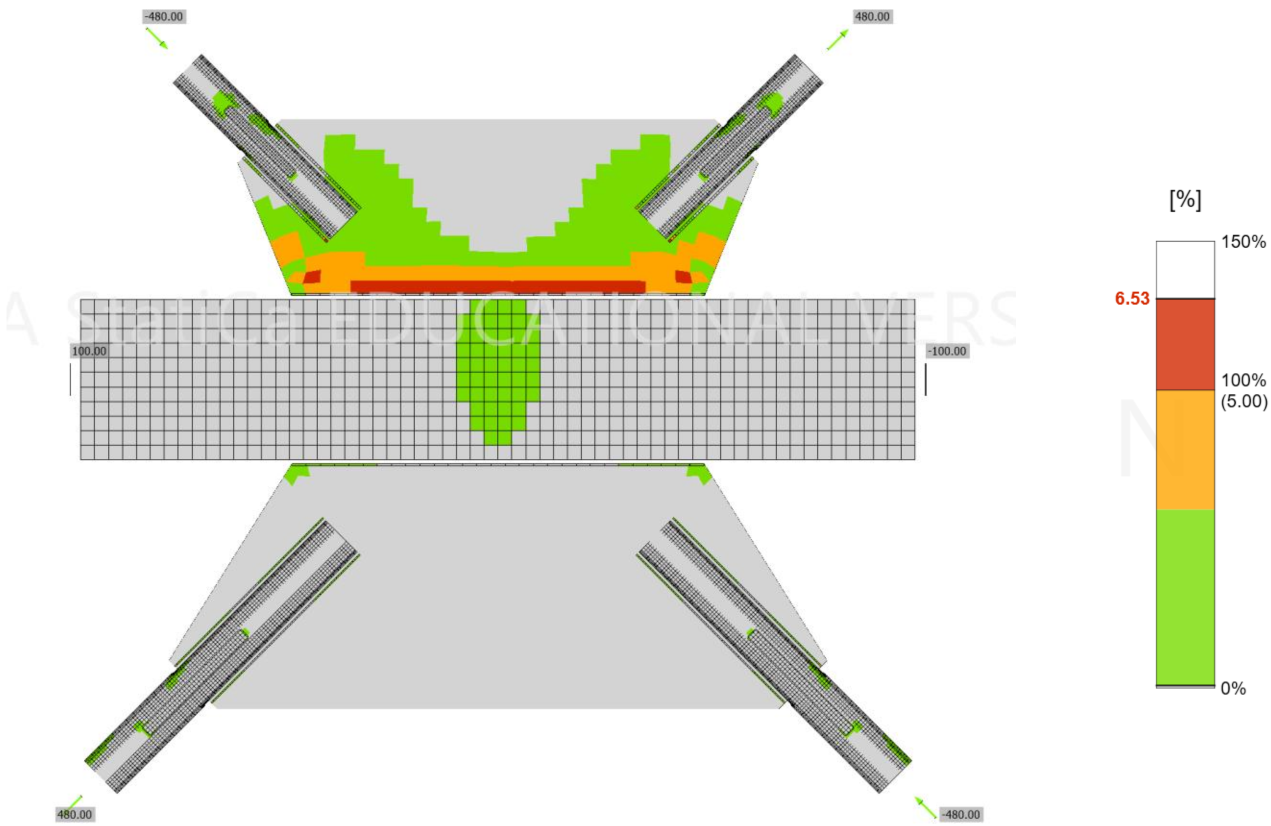
Figure 9 - von Mises stresses in connection with the action of design loads in brace - CBFEM

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

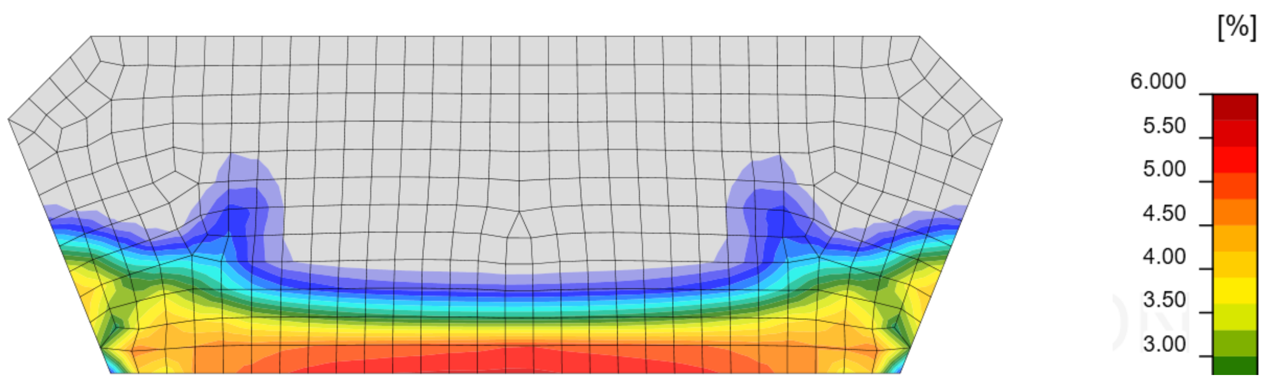
The calculations according to AISC were performed in accordance with the Load and Resistance Factor Design (LRFD) procedure to evaluate the different limit states in the connection. In the case of CBFEM, these limit states were investigated individually by several iterations by monitoring the plastic strains and equivalent stresses

For the evaluation of limit state of tensile yielding of gusset plate above beam, the capacity is observed to be in CBFEM as 480kips, as shown in Figure 10. This value is 13.33% higher than that obtained from AISC is 420kips. For the case of the gusset plate below the beam, its capacity in CBFEM is observed to be 685kips which is 7% higher when compared with 638kips obtained from AISC.

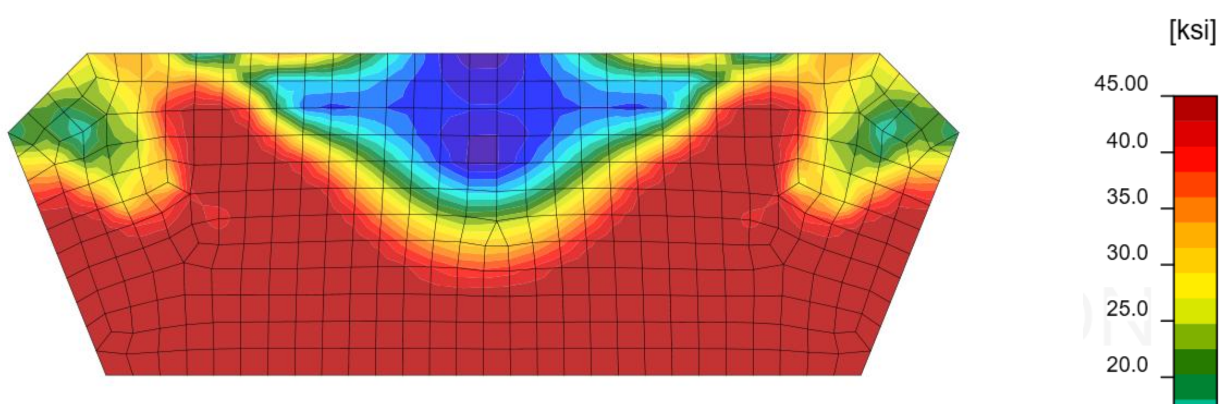
It can be observed that the **block shear limit state** at certain load exists in some members and not in others in CBFEM. For the block shear rupture in gusset plate above beam, 500kips capacity was observed in CBFEM as shown in Figure 11 which is 3.2% higher when compared to 484kips capacity as obtained from the AISC procedure.



a) Strain check in CBFEM [%]

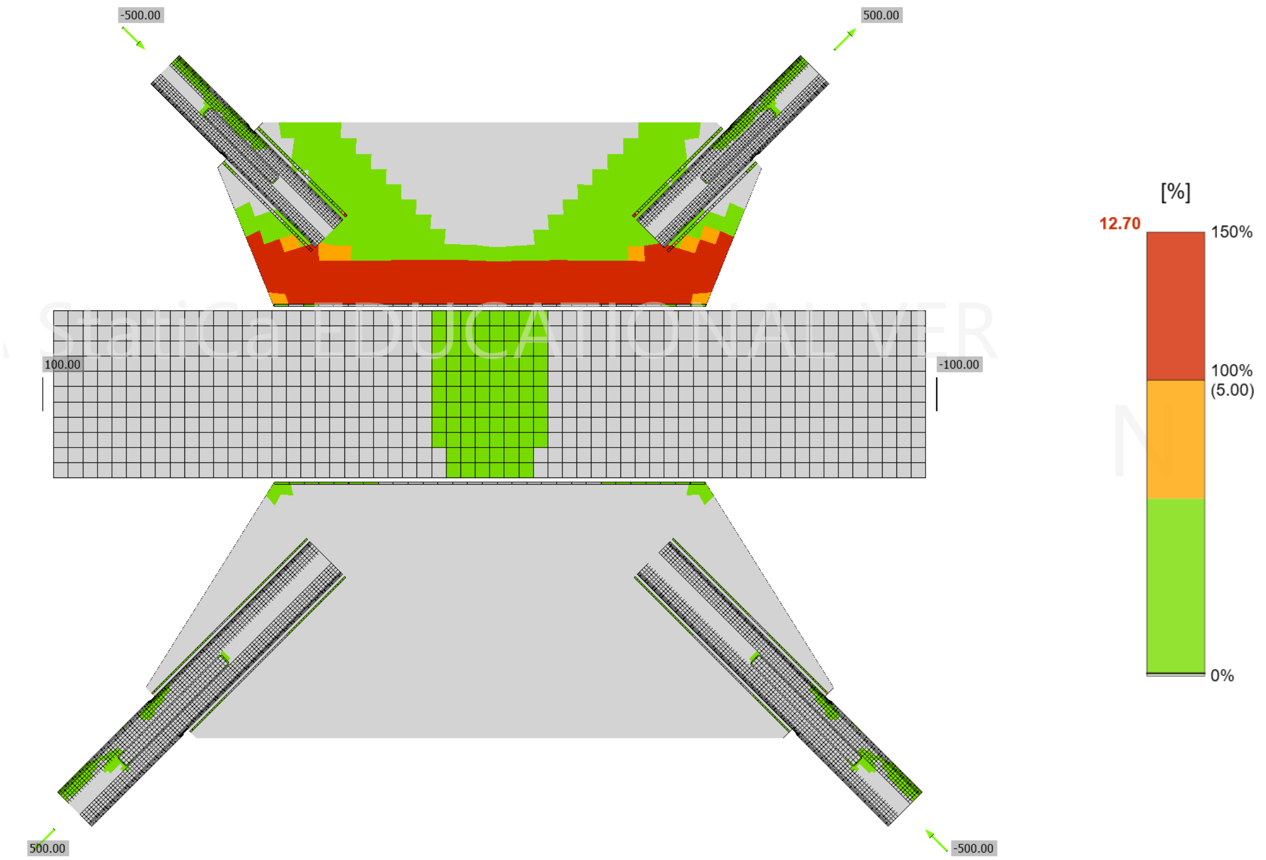


b) Check of plastic strain in CBFEM for gusset plate above beam [%]

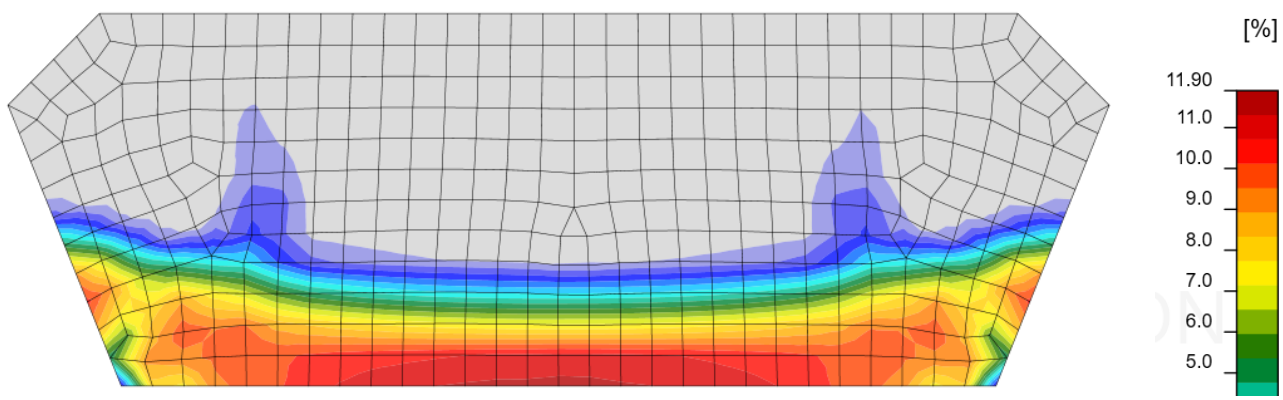


c) Check of von-Mises stresses in CBFEM for gusset plate above beam [ksi]

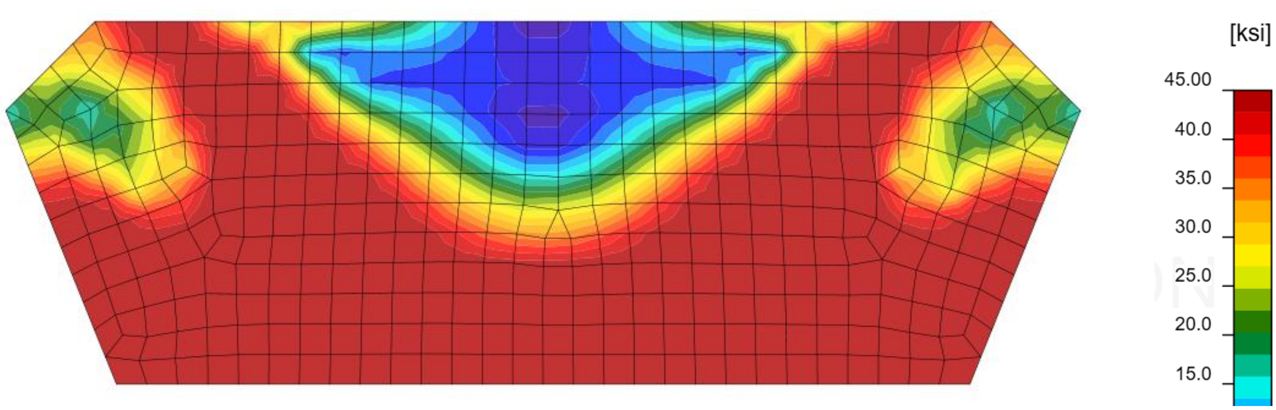
Figure 10 – Evaluation of limit state of tensile yielding of the gusset plate -CBFEM



a) Strain check in CBFEM [%]



b) Check of plastic strain in CBFEM for gusset plate above beam [%]



c) Check of von-Mises stresses in CBFEM for gusset plate above beam [ksi]

Figure 11 – Evaluation of limit state of block shear of the gusset plate – CBFEM

A typical example of weld strength evaluation according to CBFEM is shown in Figure 13. The braces are loaded such that the weld utilization percentage at the required weld is at least 100%. Then, based on the obtained weld angle from CBFEM, weld strength is calculated for the entire weld length as provided in the design.

Overall, CBFEM provides same weld size at all locations for stress-strain analysis, when compared with values from AISC as observed from Table 1,

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16^{ths} weld size (in)	
			AISC	CBFEM
1	290	Reinforcing plate and brace – Above Beam	5	5
2	344	Reinforcing plate and brace – Below Beam	5	5
3	385	Gusset Plate and Brace – Above Beam	4	4
4	405	Gusset Plate and Brace – Below Beam	4	4
5	513	Gusset Plate to Top Flange of Beam	4	4
6	585	Gusset Plate to Bottom Flange of Beam	7	7

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

Table 1: Comparison of fillet weld size as per AISC and CBFEM

The force, F_n, and weld angle, ϕ, are derived from stresses $\sigma_{\perp}$, $\tau_{\perp}$, $\tau_{\parallel}$, length, and effective area of the weld finite element. These stresses are from IDEA StatiCa. The weld capacity at several locations of the connection obtained from CBFEM are like the capacities obtained from AISC, but on a slightly conservative side.

The geometric angle of the CBFEM model which is based on FEM equations differs from the angle of loading measured from the weld longitudinal axis as output by IDEA StatiCa. These differences occur because welds are divided into short segments when modeled in IDEA StatiCa. Unlike traditional calculations, where the demands along the length of the weld are assumed to be uniform, the weld segments experience different demands based on the stiffness and poisons ratio of the weld and the connecting elements. The angle output obtained from IDEA StatiCa is usually based on the weld segment that has the greatest utilization ratio. Often, this is a segment at the end of a weld in CBFEM.

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

The **check for buckling for this connection is performed for the design compression load** in the brace, such that loads on the beam are balanced by "loads in equilibrium" in CBFEM as shown in Figure 12. Since the gusset plate is connected to the beam from one side and is connected to two braces, the buckling can be classified as "**local buckling**". The buckling factor for the critical mode is 4.43 and is observed to be more than the prescribed buckling factor of 4 for a gusset plate with this configuration and with steel grade of A572-50 subjected to local buckling.

If the buckling factor for critical model was less than the prescribed buckling factor, the thickness of the gusset plate must be increased until the minimum value of critical mode buckling factor is satisfied according to CBFEM.

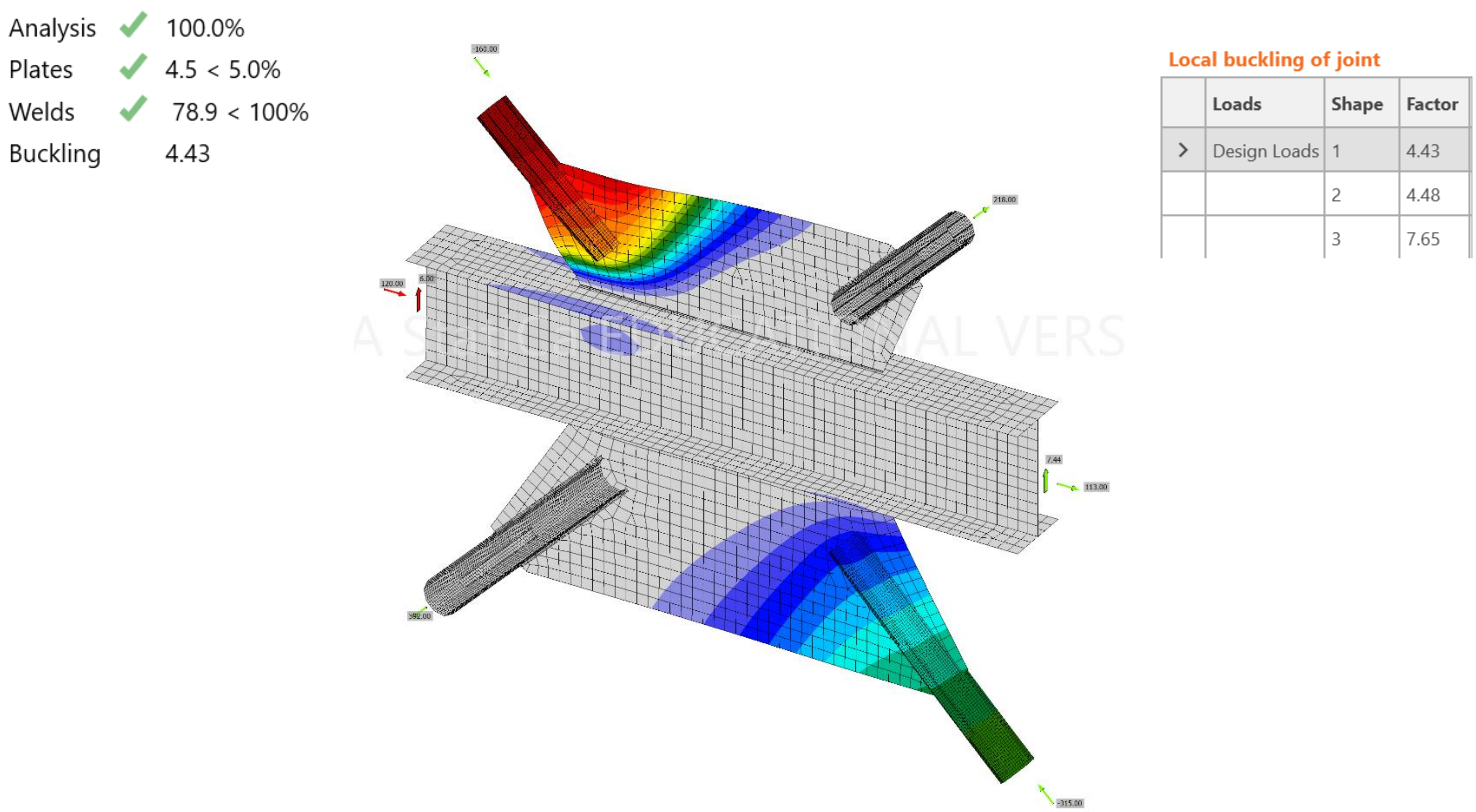


Figure 12 - Buckling analysis of a connection in two-story braced frame of steel building - CBFEM

7. Variation Studies

a. Influence of reinforcing plates on the brace members

Slot welding of HSS brace to gusset plate is one of the most common practices in this type of connection. This is intended to reduce the effective net area of the brace when subjected to the action of tension loads. To compensate for the loss of the effective net area in braces, it is essential to provide reinforcing plates on all the vertical HSS brace members. Although it is not required to provide reinforcing plates in OCBF according to AISC 341-16, it is still a recommended practice for seismic connection design. The influence of the use of reinforcing plates is evaluated in this study for an action of 215kips of axial tension in the brace as shown in Figures 13 and 14.

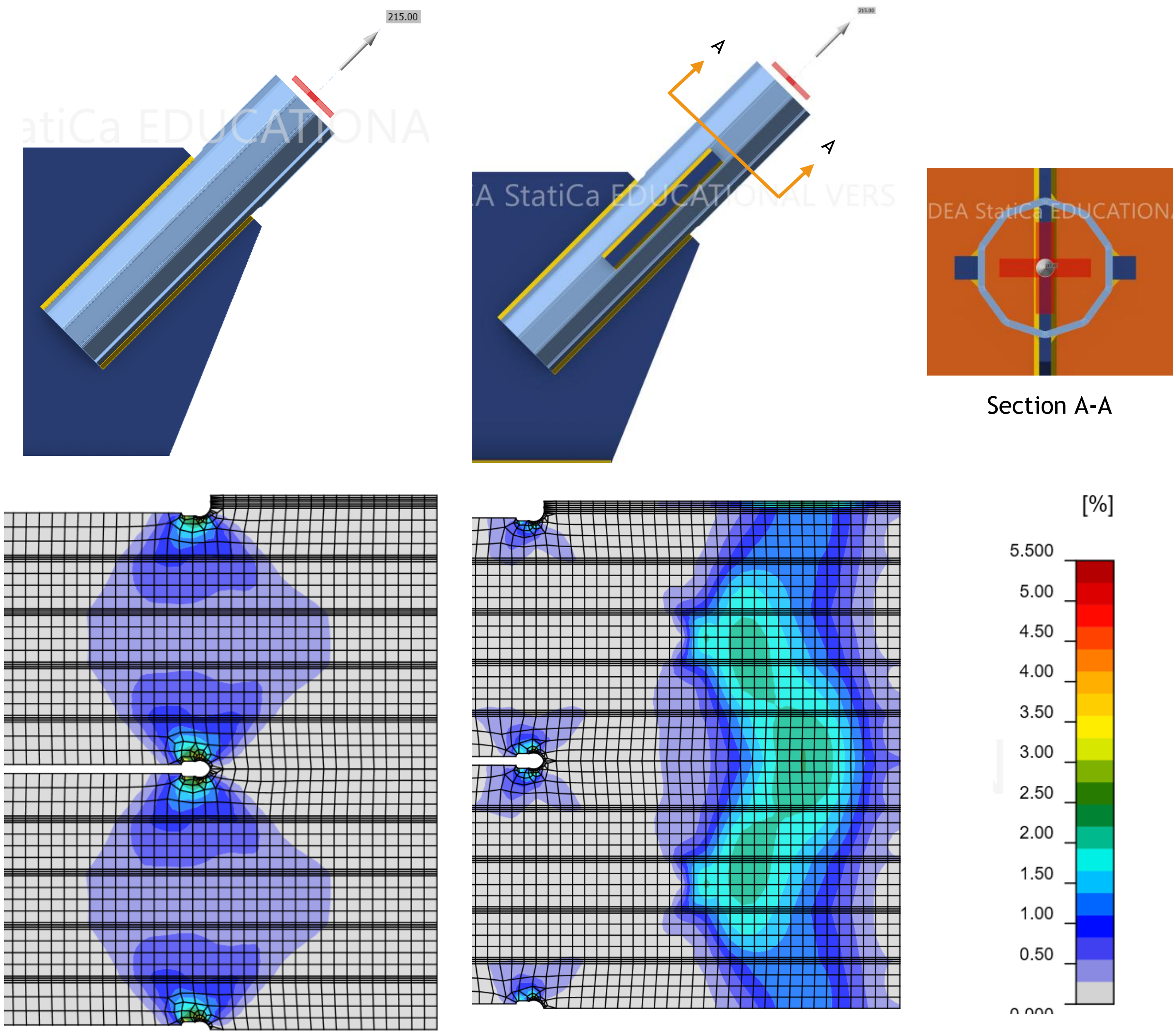


Figure 13 – Plastic strains in brace a) without reinforcement plates, b) with reinforcement plates

Based on the observations from the Figure 13 , the brace with reinforcing plates allows the inelastic demands to develop along the brace and away from the connection region. Whereas, in case of connection without the reinforcing plates, most of the inelastic demands is found to be concentrated in the net section region of the brace, which reduces the capacity of connection for tension loads.

The connection with the reinforcing plates on the brace are observed to have higher load resistance on an average by 28.5% when compared with the vertical bracing connections without reinforcing plates on the brace as shown in Figure 14.

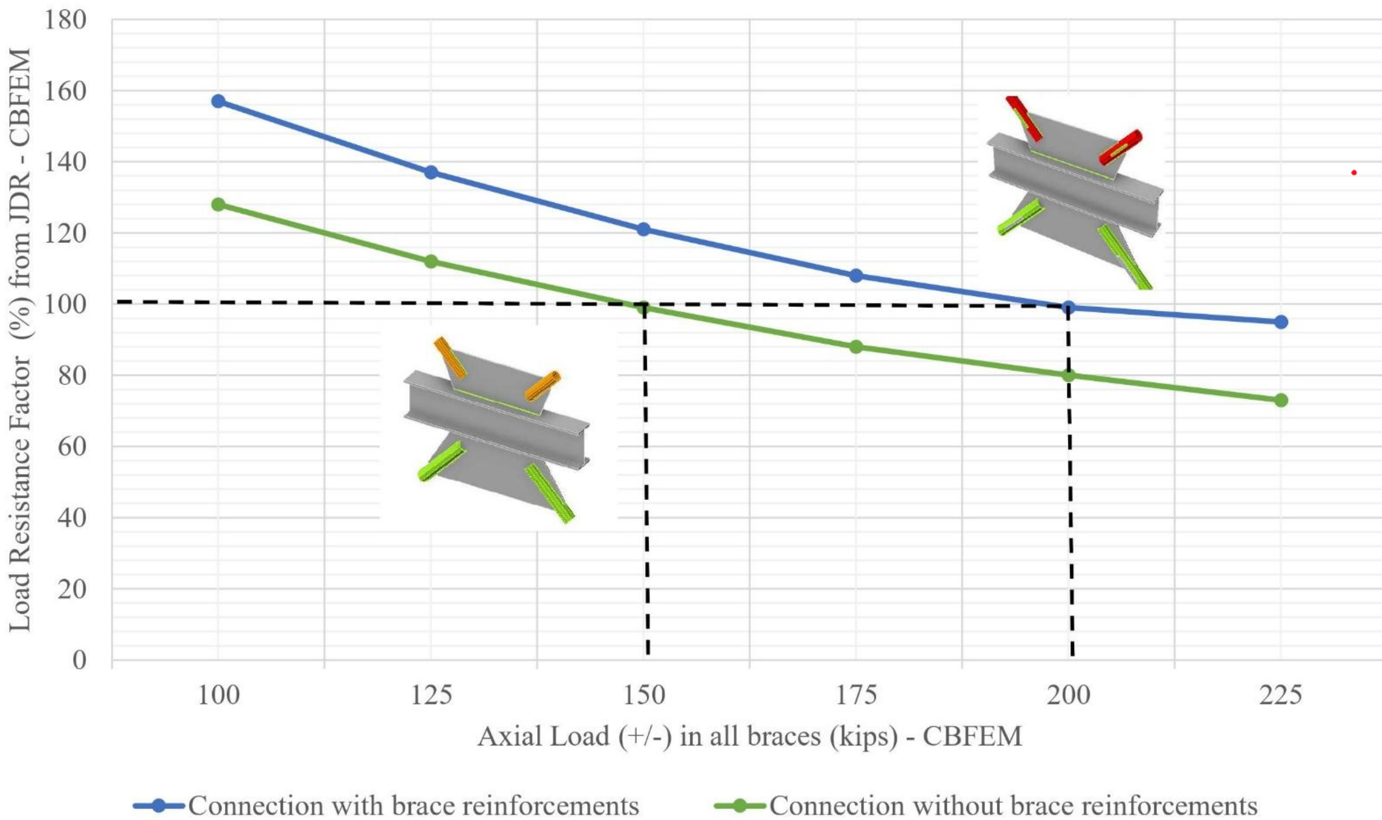


Figure 14 - Load Resistance for the vertical brace connection with and without brace reinforcements

b. Changing fillet welds at brace-to-gusset to CJP welds at all locations

The failure of fillet welds at braces was observed as one of the governing limit states in the connection to the given loads. Therefore, the fillet welds are changed in the model to Complete Joint Penetration (CJP) / butt welds. Based on this modification, the applied load at the braces can be increased to 400 kips before a limit state of weld is observed at gusset-to-beam as shown in Figure 15. Overall, a 46% increase in the load carrying capacity for the given connection is observed by changing the fillet weld to CJP.

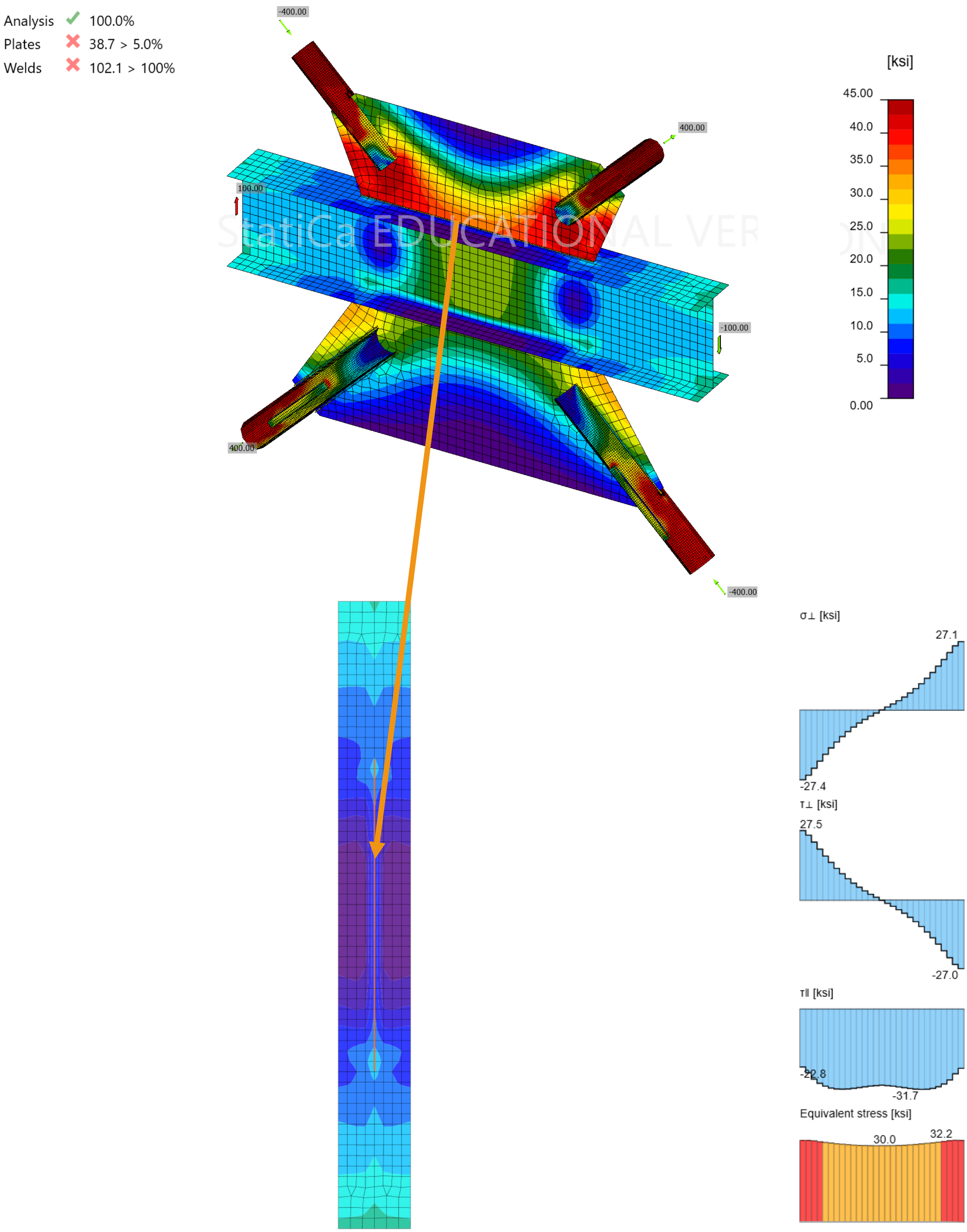


Figure 15 – von Mises stresses for change of fillet welds at all braces to gusset locations to CJP welds – CBFEM

c. Evaluation of Buckling capacity as per CBFEM

To evaluate the buckling of the connection in detail, a variation study for four different gusset plate thicknesses was performed in CBFEM. The results are tabulated in Table 2 and the plot of results is shown in Figure 16 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	HSS6x0.312	183	160	2.25	90	Gusset plate	NOT OK
2	1/2	HSS6x0.312	291	160	4.32	173	Gusset plate	OK
3	¾	HSS6x0.312	495	160	7.73	310	Brace member	OK
4	1	HSS6x0.312	690	160	3.6	374	Brace member	OK

- 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 $P_{b\_CBFEM} = (P_{comp} x\ \alpha_{cr})/4$.

Table 2 – Parametric Study for buckling of top gusset plate for multi-story X-brace connection

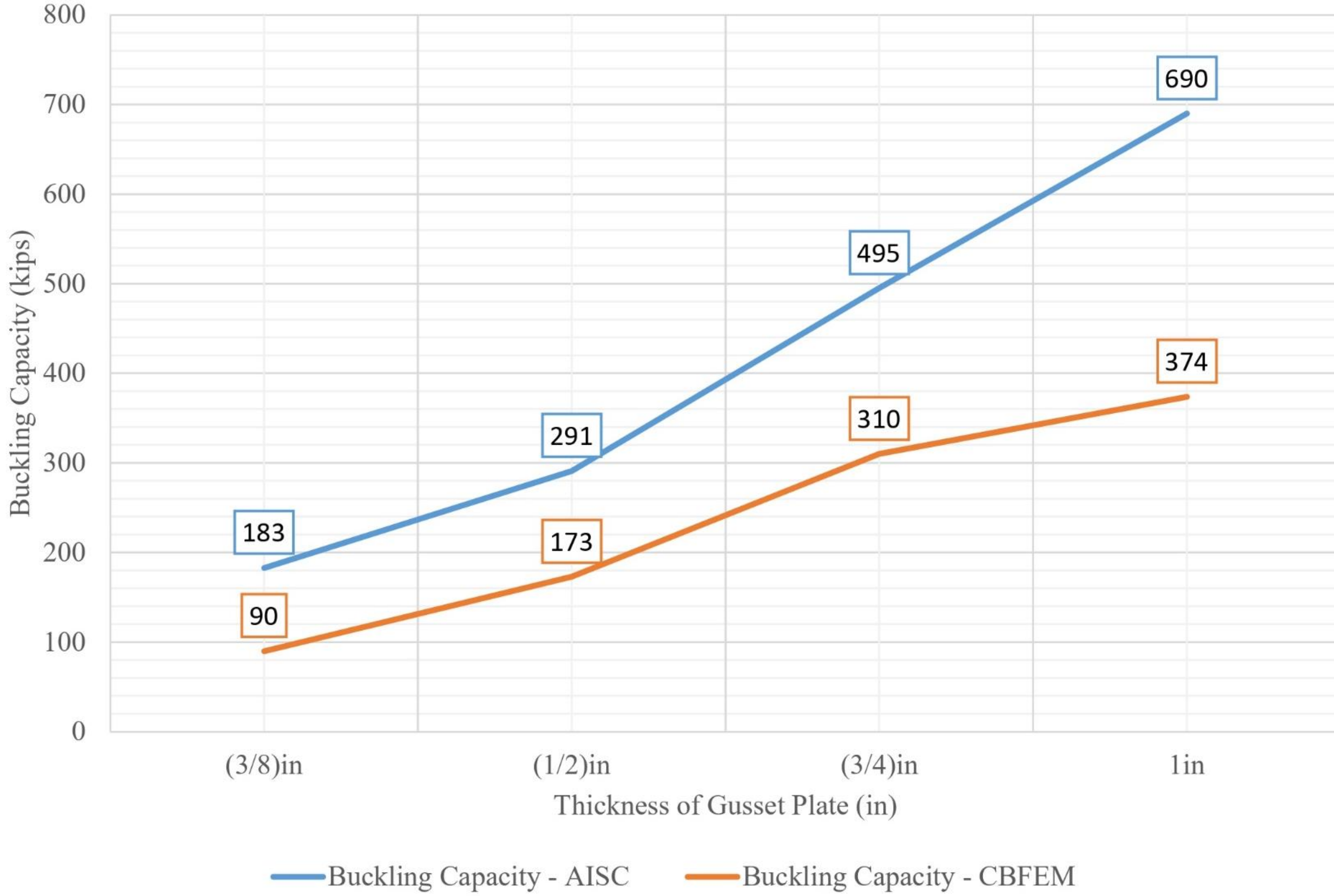


Figure 16 – Curve of Buckling Analysis vs thickness of gusset plate

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

9. Sr No	Limit State of	AISC (kips)	CBFEM (kips)	Δ_{AISC_CBFEM} (%)
1	Tensile Yielding of Gusset Plate – Above Beam	420	480	13
2	Block Shear Rupture of Gusset Plate – Above Beam	484	500	3.5
3	Tensile Yielding of Gusset Plate – Below Beam	638	685	7

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16ths weld size (in)	
			AISC	CBFEM
1	290	Reinforcing plate and brace – Above Beam	5	5
2	344	Reinforcing plate and brace – Below Beam	5	5
3	385	Gusset Plate and Brace – Above Beam	4	4
4	405	Gusset Plate and Brace – Below Beam	4	4
5	513	Gusset Plate to Top Flange of Beam	4	4
6	585	Gusset Plate to Bottom Flange of Beam	7	7

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

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.
3. The gusset plate limit states including yielding and tension rupture are based on the 5% plastic strain limit in CBFEM.
4. 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 rupture strength which is based on the ultimate strength of steel.
5. 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.
6. 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.
7. Weld capacity as computed by CBFEM is similar to that as per AISC. The angle output which is provided by IDEA StatiCa is usually based on the weld segment that has the greatest utilization ratio. Often this is a segment at the end of a weld in CBFEM.

11. References

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