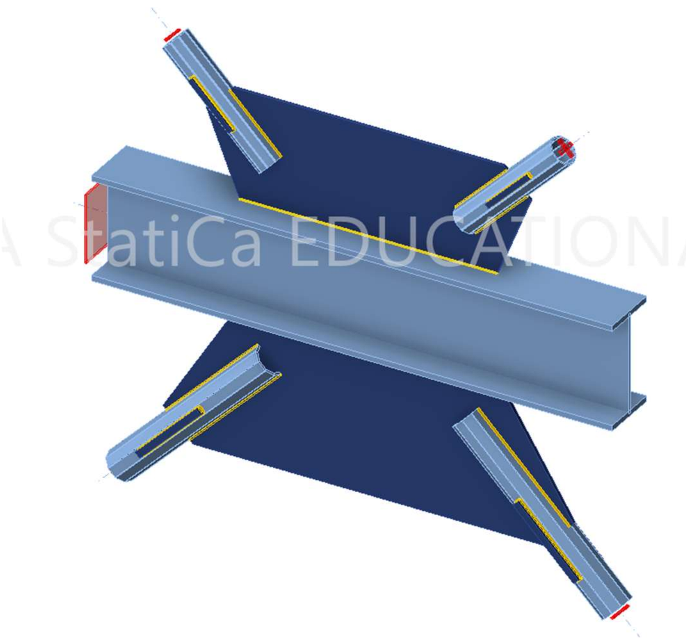
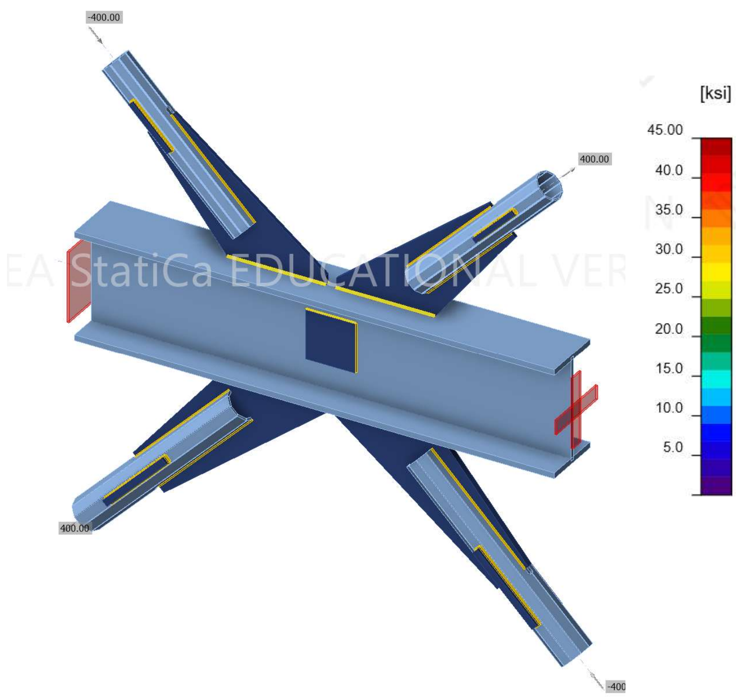
Connection in a Two-Story X-Brace Configuration of Special Concentrically Braced Frame (SCBF) System





This is the third verification example in CBFEM from a series of seismic vertical brace connections wherein a connection in multistorey X-brace configuration of special concentrically braced frame (SCBF) system is evaluated according to a procedure from Seismic design manual (AISC 341-16) and CBFEM method in IDEA StatiCa connection Version 23.0.

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1. Problem Description

The objective of this example is to verify the component-based finite element method (CBFEM) for a connection in two story X-braced configuration of a special concentric braced frame (SCBF) in a steel building. The floor plan, elevation and connection details as provided in example 5.3 of AISC341-16 are considered as shown in Figure 1 and 2 respectively. The results obtained from calculation methods which are based on Steel Construction Manual, Fourteenth Edition (AISC 360, 2016) and Seismic Design Manual, Third Edition (AISC 341, 2016) are then compared with results obtained from the CBFEM using capacity design analysis in IDEA StatiCa software version 23.0.

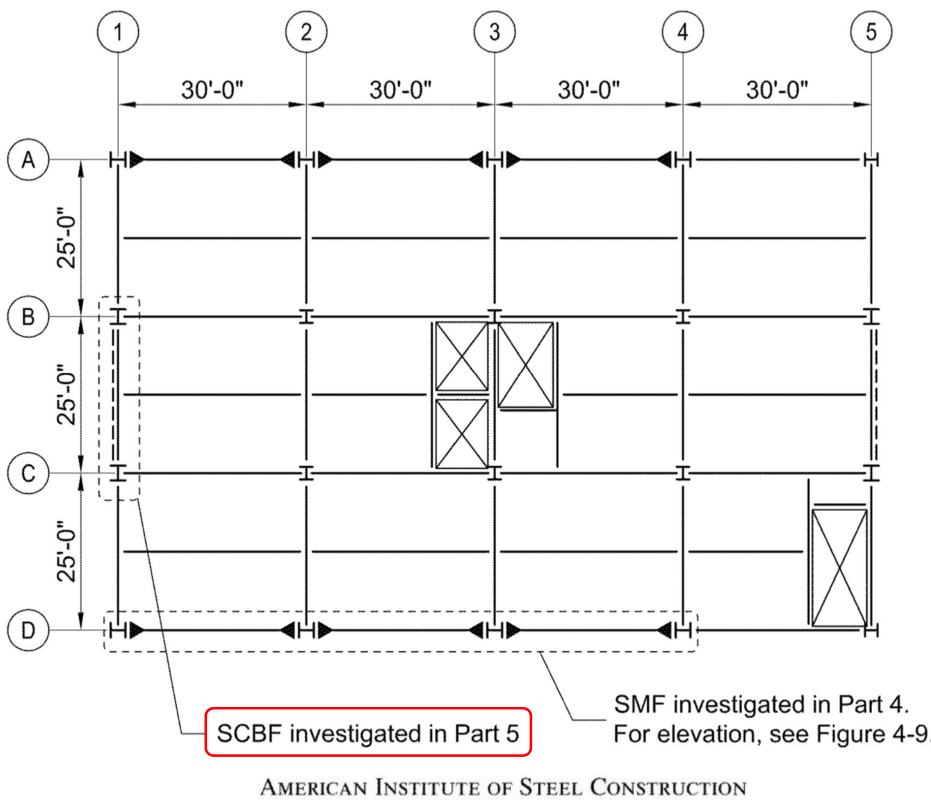

Figure 1 - Floor Plan of steel building highlighting the investigated SCBF (AISC 341, 2016)

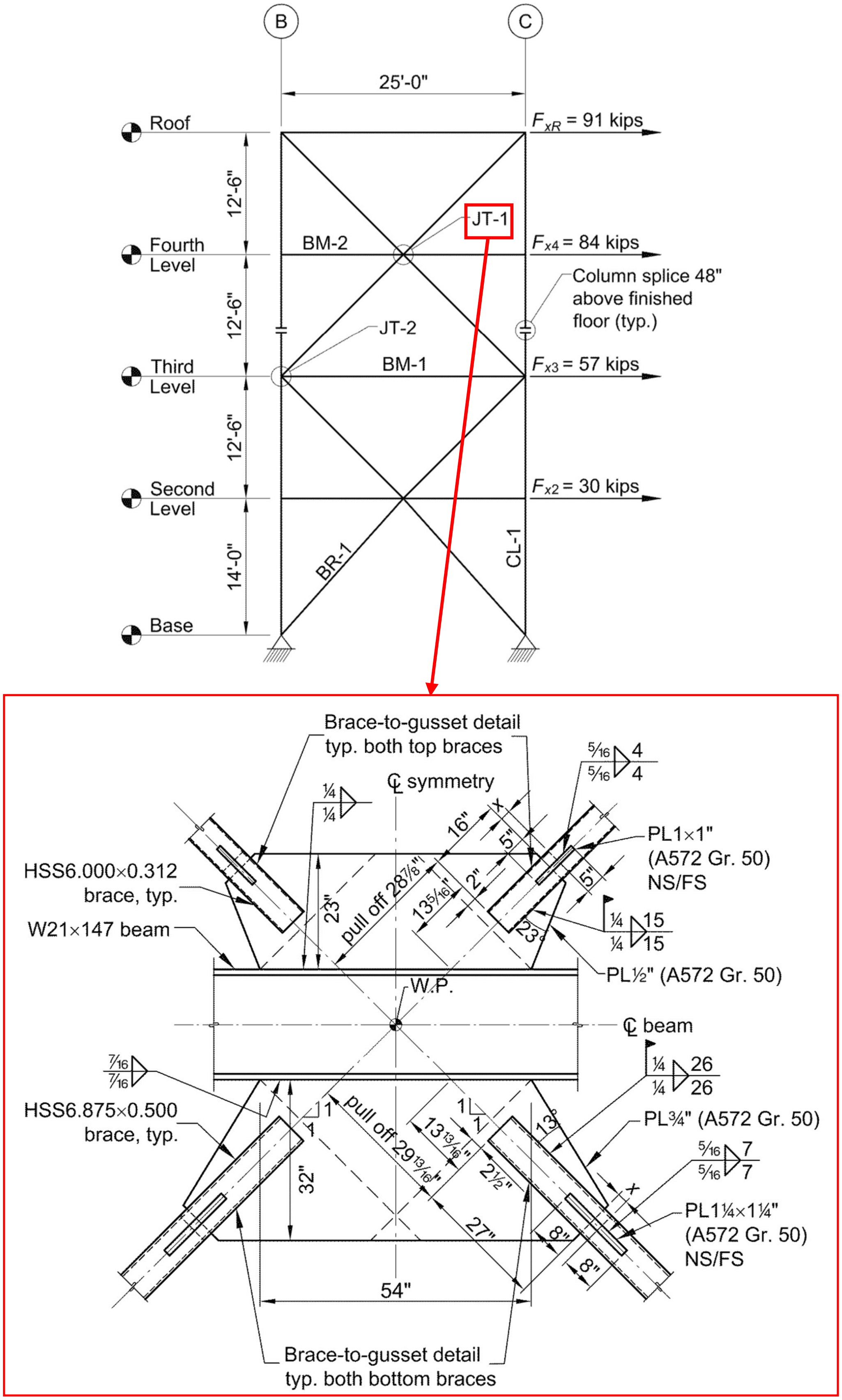

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Figure 2 - Braced frame elevation view and considered connection details (AISC 341, 2016)

The presented verification design example is referred from example 5.3.7 of seismic design manual, third edition (AISC 341-16), and the details for the member as well as connection are as presented 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

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

Connection Details

1. Welds (Fillet welds at all applicable locations)

- 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.
- 1/4” rear sided fillet weld between bottom brace 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 connection modelling in IDEA StatiCa begins with defining the members- one beam and four braces. Then, the two gusset plates and eight reinforcing plates on braces are defined and arranged as per connection details. A beam is assigned as a bearing member in CBFEM and loads on it are balanced by "loads in equilibrium" after the brace loads are defined using stress-strain analysis method.

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 reinforcing plates and braces. The details for the HSS braces in the top gusset plate are as shown in Figures 3.

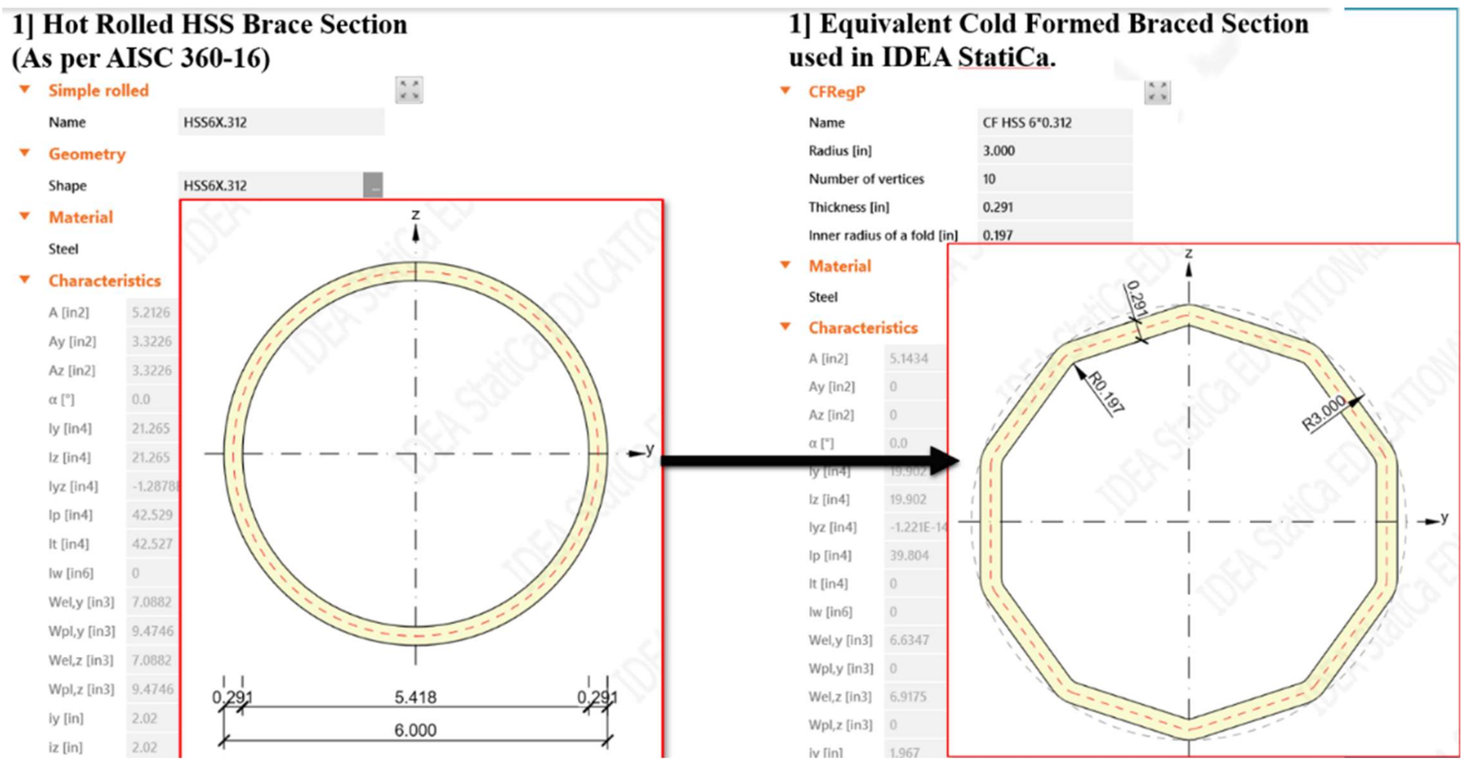


Figure 3 - Brace members as equivalent cold-formed sections in IDEA StatiCa - CBFEM

To perform the CBFEM analysis of the given connection, it was modeled in the IDEA StatiCa Version 23.0 using several operations in software, as shown in Figure 4, and a base model was prepared.

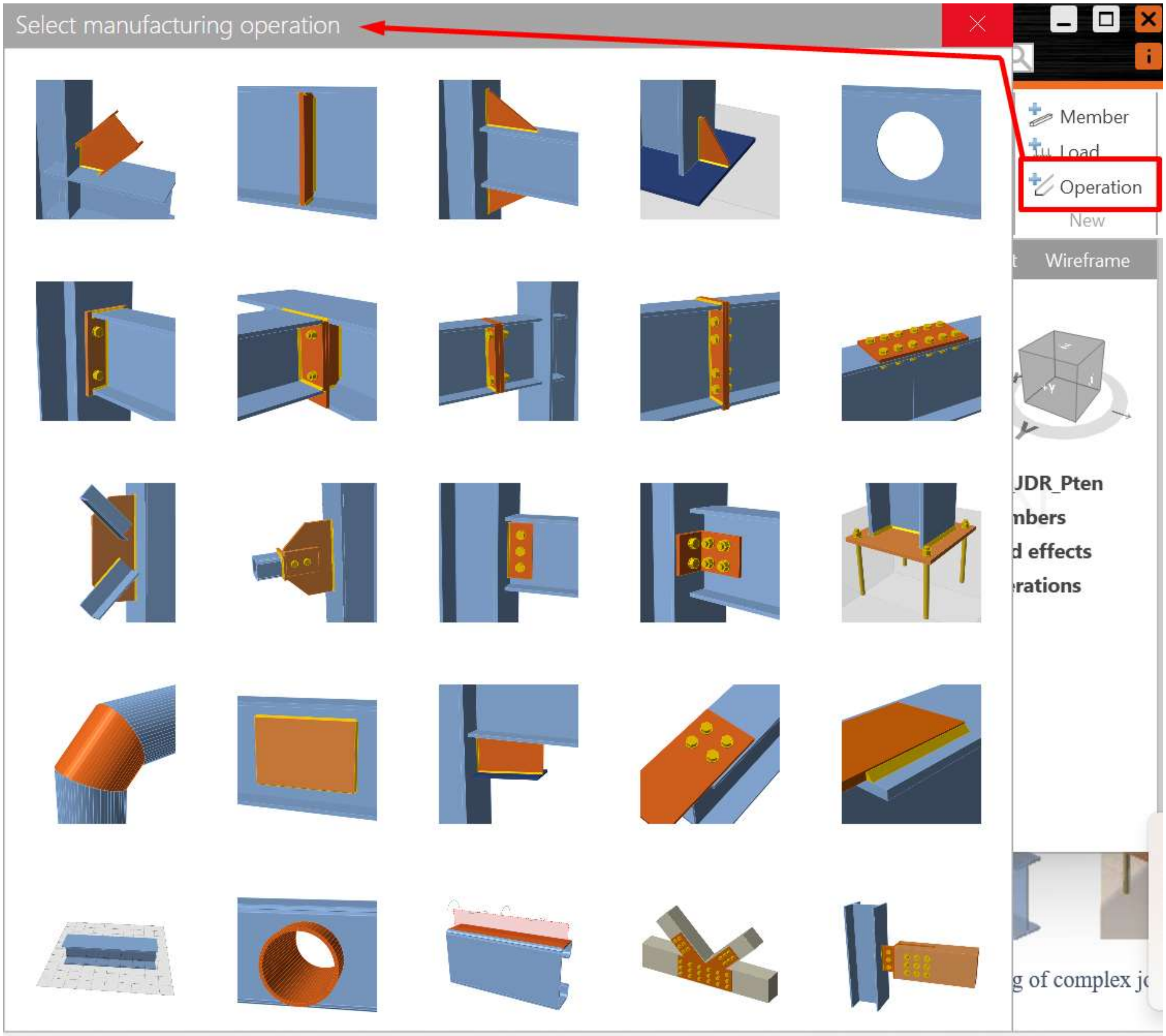


Figure 4 - Several manufacturing operations in IDEA StatiCa for connection modelling - CBFEM

Typical HSS bracing connections utilize a knifed gusset plate, resulting in a reduced section immediately adjacent to the gusset plate. While this net-section area is observed to be a weak link in brace behavior, the reduction in ductility was never quantified. Several performed research demonstrated that typical designs that do not have reinforcing plates on the brace member, as shown in Figure 5, were not ductile, whereas reinforced connections performed well for seismic loads. The seismic design provisions then mandated reduced sections to be designed to be stronger than the brace, allowing distributed brace yielding along the brace.

Figure 5 – Vertical seismic HSS brace connections without the reinforcing plates on the brace member (AISC 341, 2016)

Typical seismic connection details as shown in Figure 6 now include reinforcement plates and conform to the requirements of Section F2.5b(c) of AISC 341-16. (Roeder et al., 2011; Simpson & Mahin, 2018; Sizemore et al., 2017).

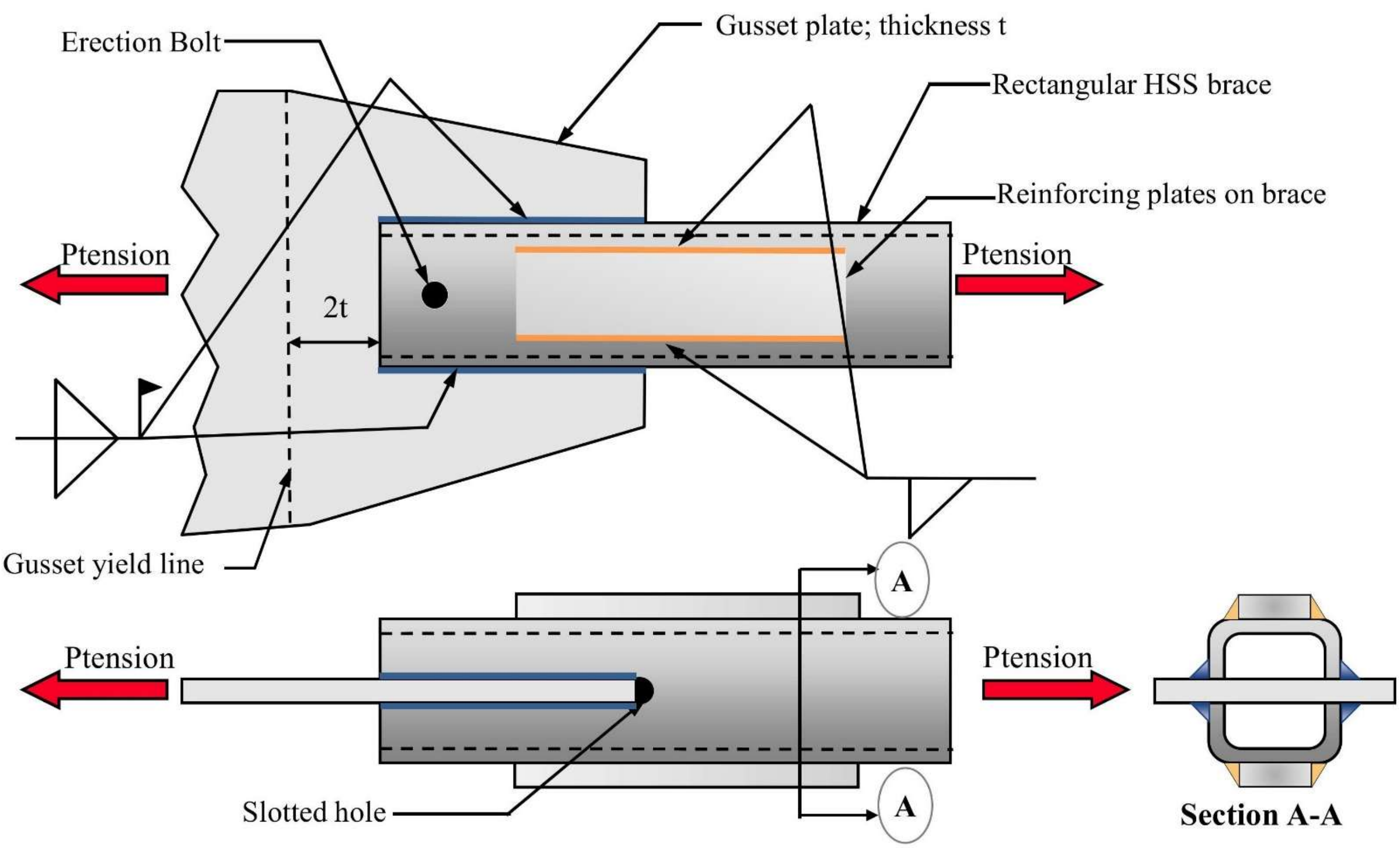
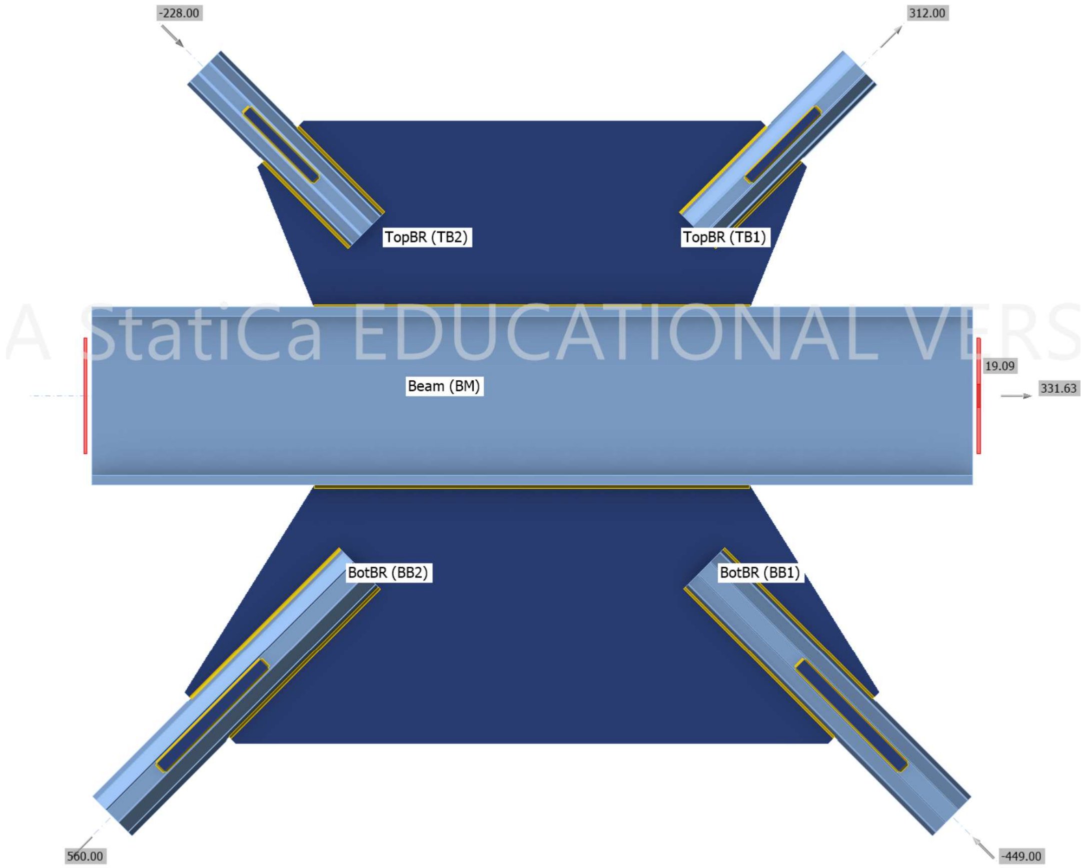
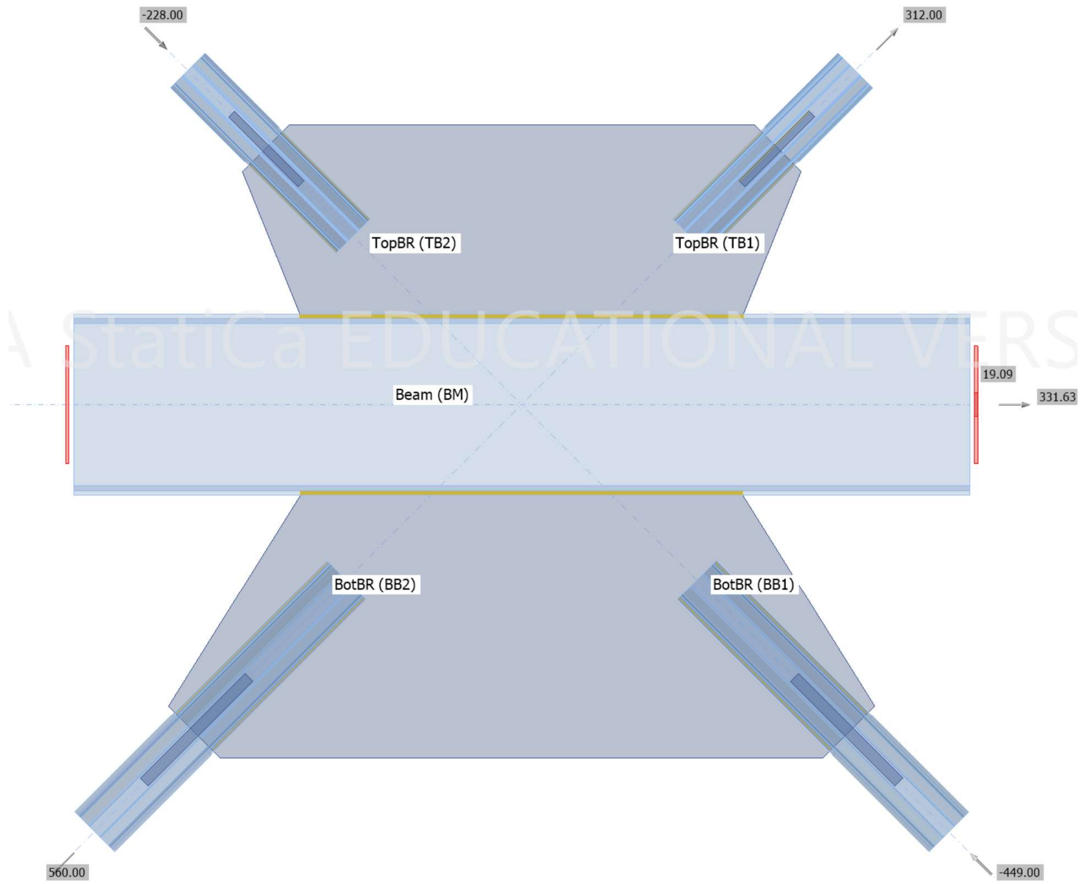


Figure 6 – Typical HSS seismic brace connection details with the reinforcing plates on the brace member

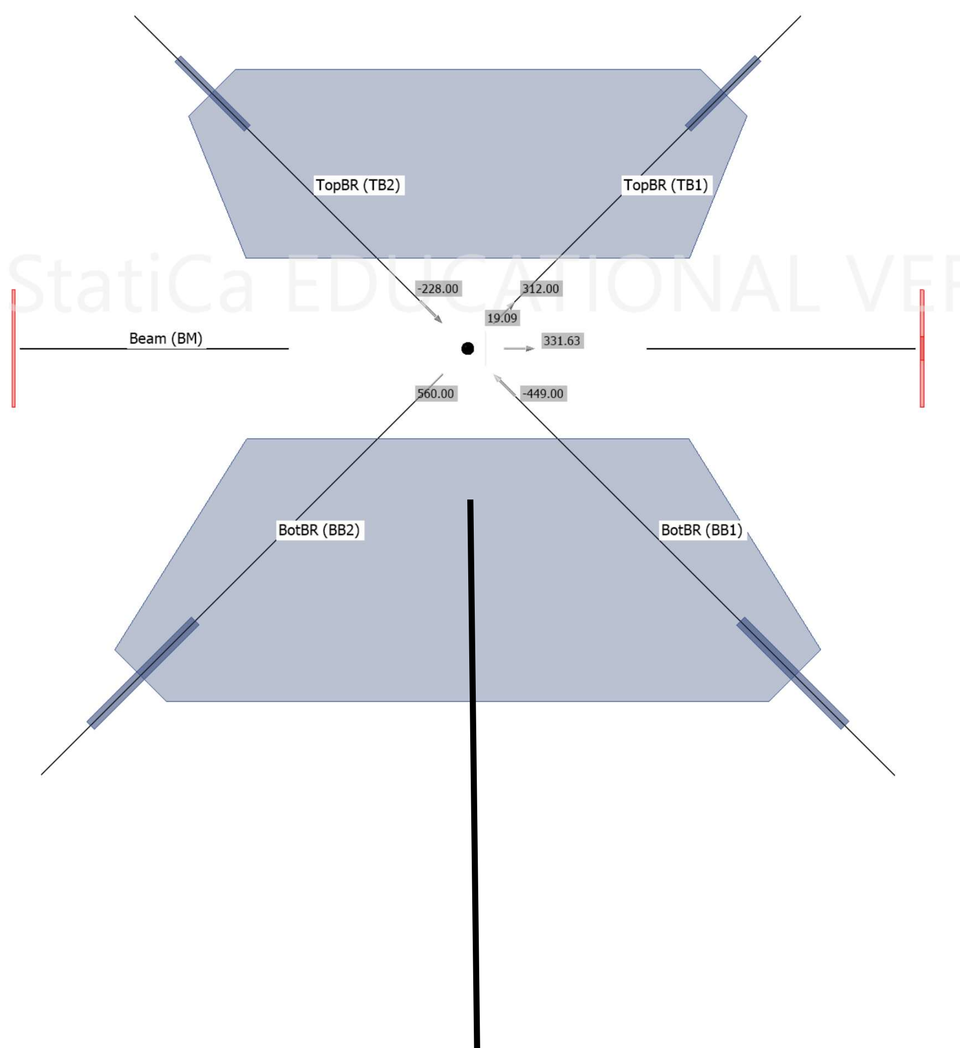
The model type of N-V_y-V_z-M_x-M_y-M_z (Fixed) is assigned to beam, while model type of N-V_y-V_z (Pinned) is provided to all the four bracing members, such that the forces are in the node, as seen in wireframe view in Figure 7.

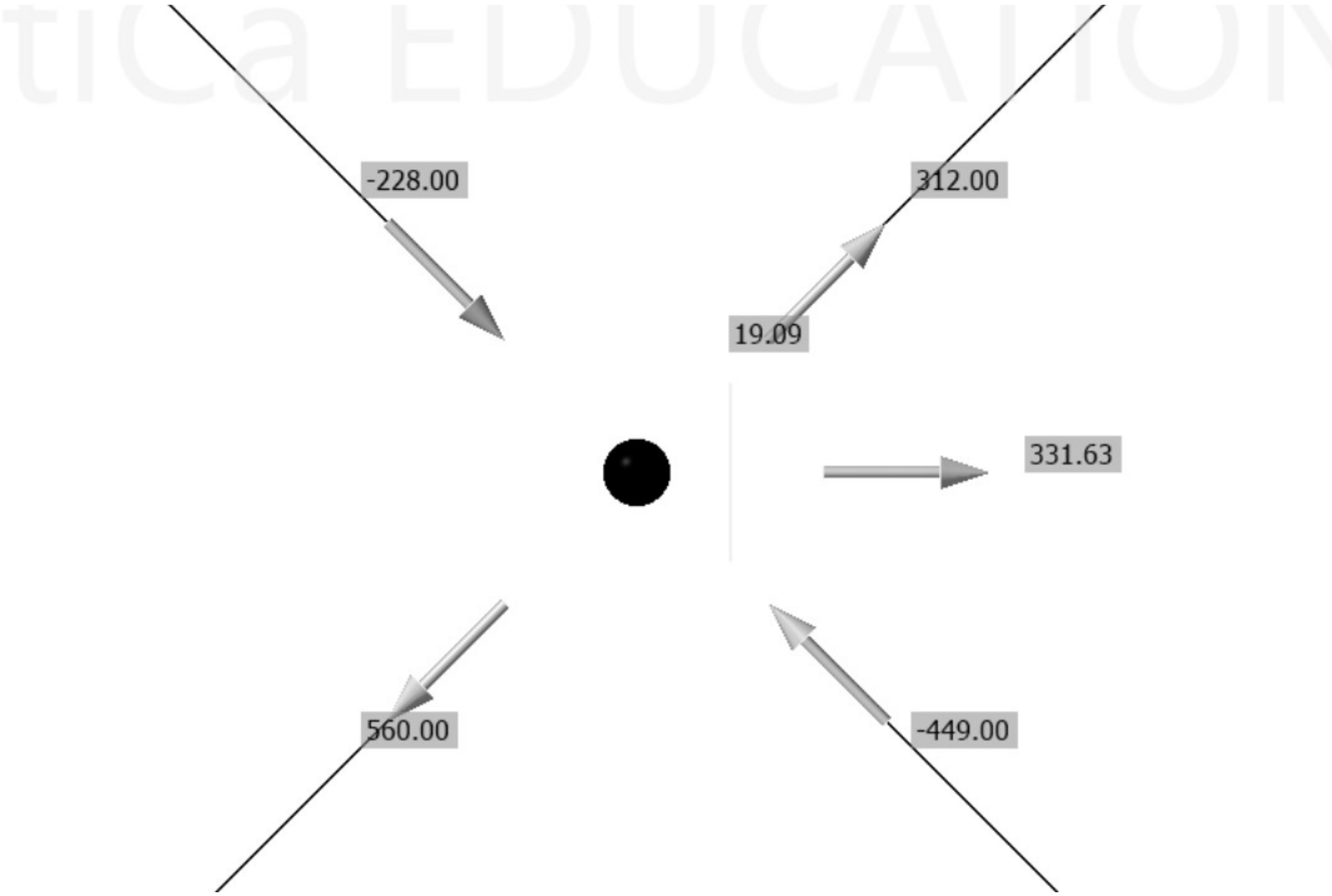


a) Solid view of base model in CBFEM



b) Transparent view of base model in CBFEM





c) Wireframe view of base model in CBFEM

Figure 7 - Configurations of the view of the connection for SCBF – Solid, Transparent and Wireframe view

Visualization of connection or member failure can be easily predicted 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 8,

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 8 - Visualization of failure in overall check for connection and member - CBFEM

Based on the observation for the given connection from Figure 8, conclusions are tabulated in Table 1 below.

Table 1 – Traffic light type format visualization of the connection in CBFEM

Sr No	Color	Elements	Comments
1	**GREY**	Not Applicable.	Elements are safe for action of loads.
2	**GREEN**	Beam, Bottom brace members, reinforcing plates on the bottom brace members, Gusset plate below beam, all the welds for connecting elements below beam.	Elements are safe for action of loads.
3	**ORANGE**	Reinforcing plates on the top brace members.	Elements are on verge of failure, for the applied loads.
4	**RED**	Top brace members, Gusset Plate above beam, All the welds for connecting elements above beam.	Elements are failing for the applied loads; utilization is beyond 100%.

3. Verification of Resistance in CBFEM – Capacity of Elements (Plates and Members)

The capacity design was performed for the given connection in CBFEM. All the four brace members were selected as dissipative items. The capacity of members, gusset plates and welds were then evaluated.

The evaluation of capacity for elements – plates and members are performed. The axial loads in all the four brace members of given connection base model were applied starting from 50kips to 530kips, until the plastic strain of 5% in plates or members is reached or slightly exceeded. Based on the obtained results, a plot of axial load in all braces in CBFEM (kips) vs plastic strain in members and plates in CBFEM (%) was prepared for axial loads ranging between 260 kips and 530 kips, as shown in Figure 9.

The equivalent plastic strain represents the plastic strain displayed by analysis results in failing element, while the maximal equivalent plastic strain [ε_{pl}] represents the plastic strain which was observed to be present in the failing element, hence was observed to be governing one. The same can be confirmed when viewing the results in strain check, as shown in Figures 12 and 13.

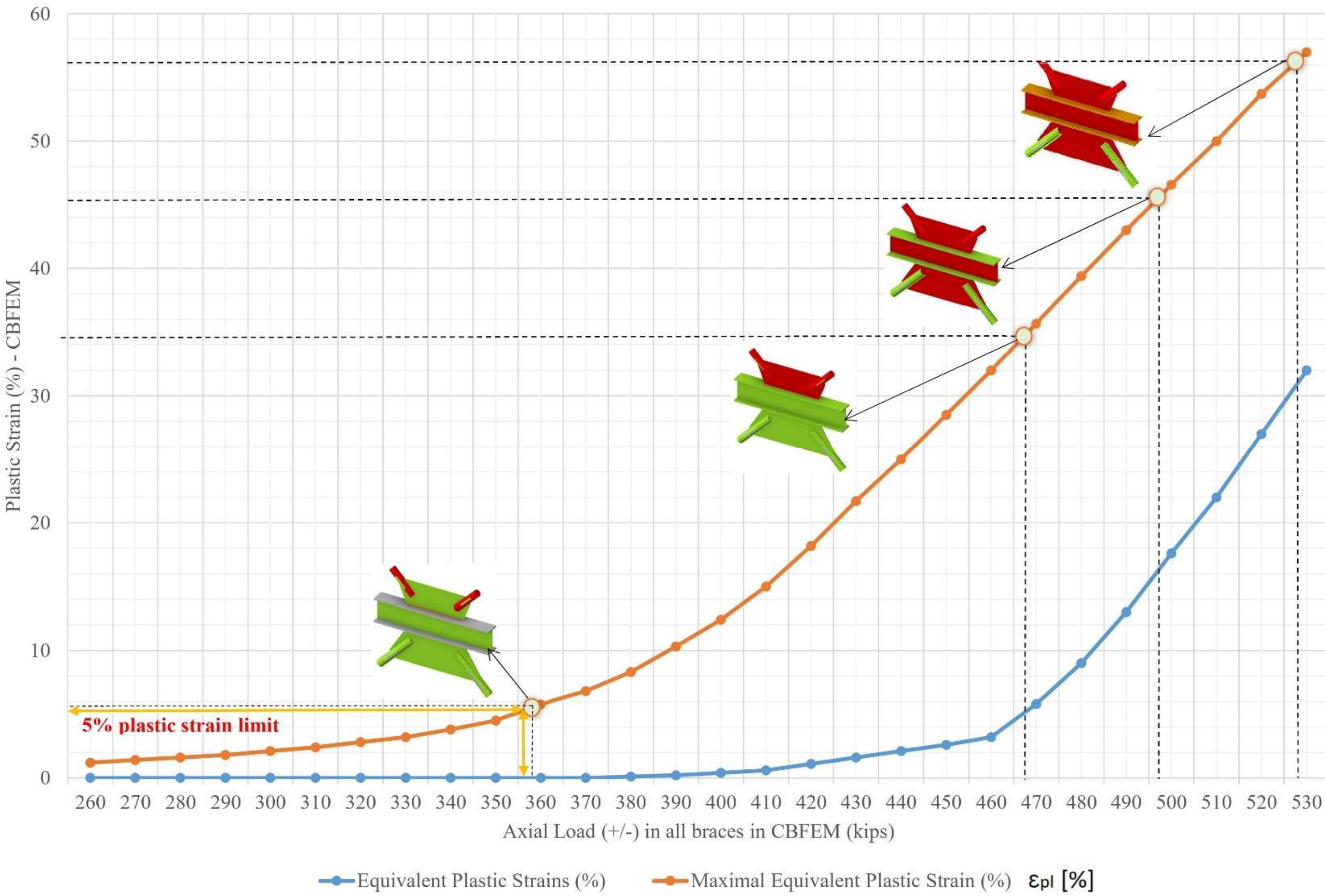


Figure 9 - Graph for evaluation of capacity of elements – CBFEM

Based on the observations, for an axial load (tension or compression) of 358kips in braces, plastic strain of more than 5% was reached, and was observed in the top braces. Similarly, for the axial load of 468kips, failure of top gusset plate was observed, such that the plastic strain in the gusset plate is 5% and maximal equivalent plastic strain is around 35%. For the case of axial load of 498kips, failure of bottom gusset plate, along with the failure of beam was observed for which the maximal equivalent plastic strain is 46%. Finally, for an axial load of

530kips, both flanges of beam are observed to fail and maximal equivalent plastic strain corresponding to the same was 56% in CBFEM.

Table 2 – Capacity of elements as per traffic light type format visualization of the connection

Sr No	P_{TorC} (kips)	Comments
1	358	✓ Yielding of brace members above the beam.
2	468	✓ Failure of reinforcing plates on the brace above beam. ✓ Failure of gusset plate above beam.
3	498	✓ Failure of gusset plate below beam. ✓ High plastic strains in beam web.
4	530	✓ High plastic strains in the flanges of beam.

4. Verification of resistance in CBFEM – Capacity of Connections (Welds)

For the given connection, several elements like plates and members are connected to each other through the welded connections (fillet welds) of different sizes and lengths. After evaluating the capacity of elements, the evaluation of welded connections for the same models and details was performed, and weld angle (θ_{CBFEM}) from CBFEM is obtained for each location, such that at least 100% weld utilization at the considered location is observed. The details for the capacity of fillet welds (ϕR_{n_CBFEM}) are presented in Table 3 below.

Table 3 – Capacity of connection – fillet welds

Sr No.	Location of welds	Data as per connection design		Results from CBFEM		
		L_{weld} (in)	D_{16} (in)	$L_{weld\_critical}$ (in)	θ_{CBFEM} (Degree)	ϕR_{n_CBFEM} (kips)
1	Connecting reinforcing plate and brace – Above Beam	12	5	0.33	24.1	335
2	Connecting reinforcing plate and brace – Below Beam	18	5	0.42	11.4	509
3	Connecting gusset plate and brace – Above Beam	15	4	0.43	21	334
4	Connecting gusset plate and brace – Below Beam	27	4	0.43	7.7	662
5	Connecting gusset plate and to top flange of beam	54	4	1.86	63.6	778
6	Connecting gusset plate and to top flange of beam	54	7	1.85	86.5	1056

- L_{weld} is the total length of weld in inches as per connection design.
- D_{16} is the weld size, which is specified to the 16^{th} of an inch.
- $L_{weld\_critical}$ is the total length of critical element of weld in CBFEM.
- θ_{CBFEM} is the weld angle in degree as obtained from CBFEM, which is based on the failure of the 1/2in critical element of weld, such that 100% weld utilization is reached for the applied tension loads in the braces.
- ϕR_{n_CBFEM} is the capacity of fillet weld in kips as obtained for CBFEM, calculated using equation J2-4 of AISC360-16

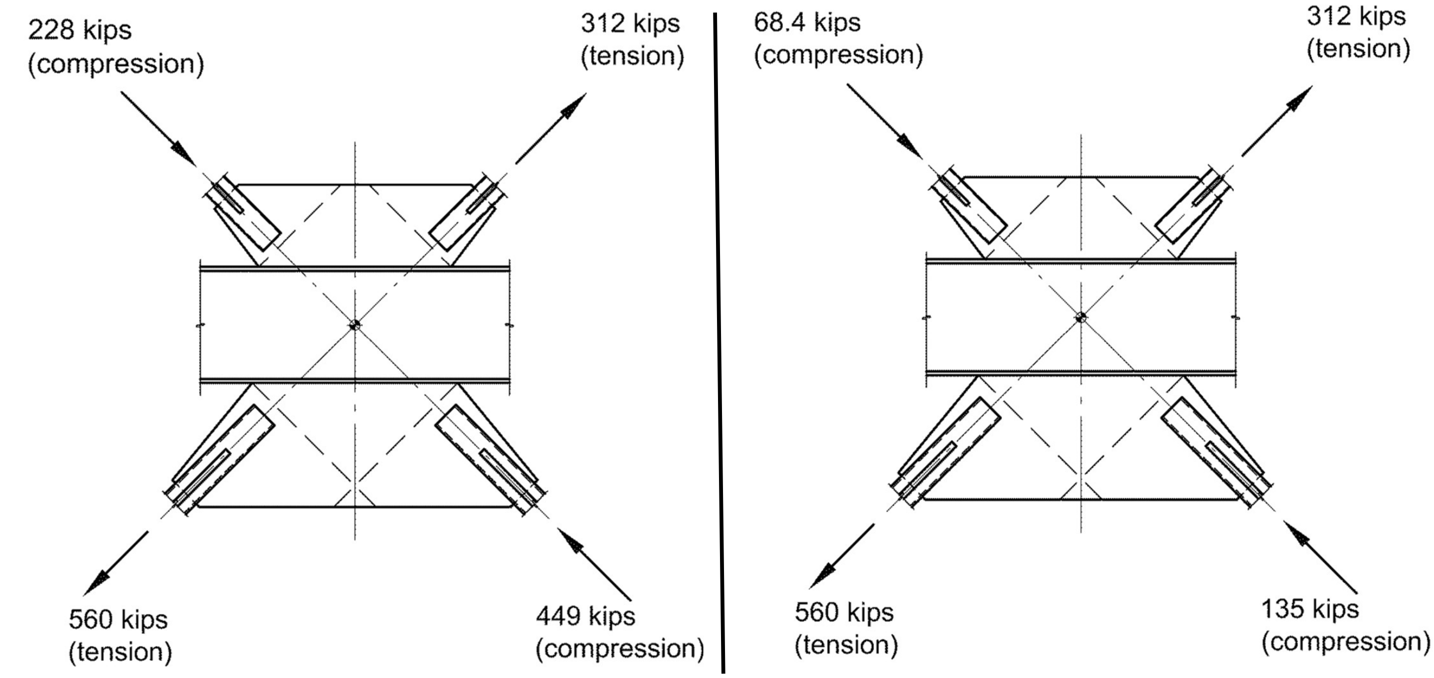
5. Code check of connection for design loads in CBFEM

Proportioning the braces to their required strength is recommended practice by AISC 341-16 for Special Concentrically Braced Frame (SCBF). As per AISC Seismic Provision Section F2.3, the required strengths of columns, beam and connections are based on the load combinations in the applicable building code, where the seismic load effect with overstrength, E_{mh} is based on the larger force determined from the following two analysis, (AISC 341, 2016)

- An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension.
- An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength.

The expected brace strength in tension is $R_yF_yA_g$, where Ag is the gross area in in^2. At the same time, the expected brace strength in compression can be taken as lesser of $R_yF_yA_g$ and $(1/0.877)F_{cre}A_g$, where F_{cre} is determined from Specification Chapter E using the equation for F_{cr}, except that the expected yield stress R_yF_y is used instead of F_y. The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression (Marcu & Köber, 2021) (Ebrahimi & Mirghaderi, 2021).

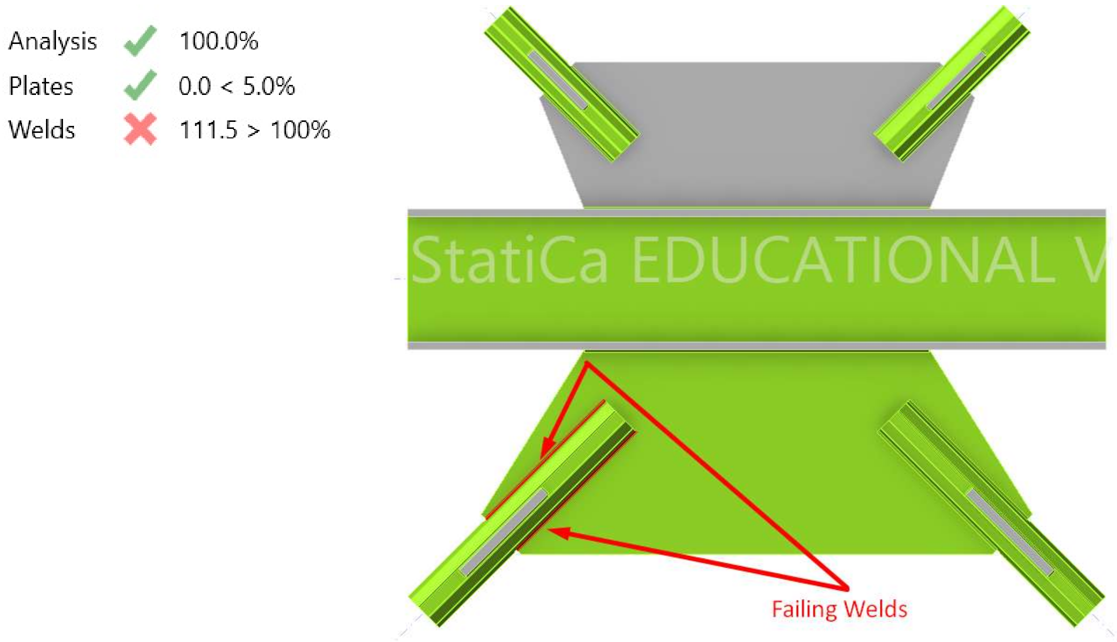
The design loads obtained for the connection for both load cases – Buckling load Case and Post-Buckling load case are as shown in Figure 10.



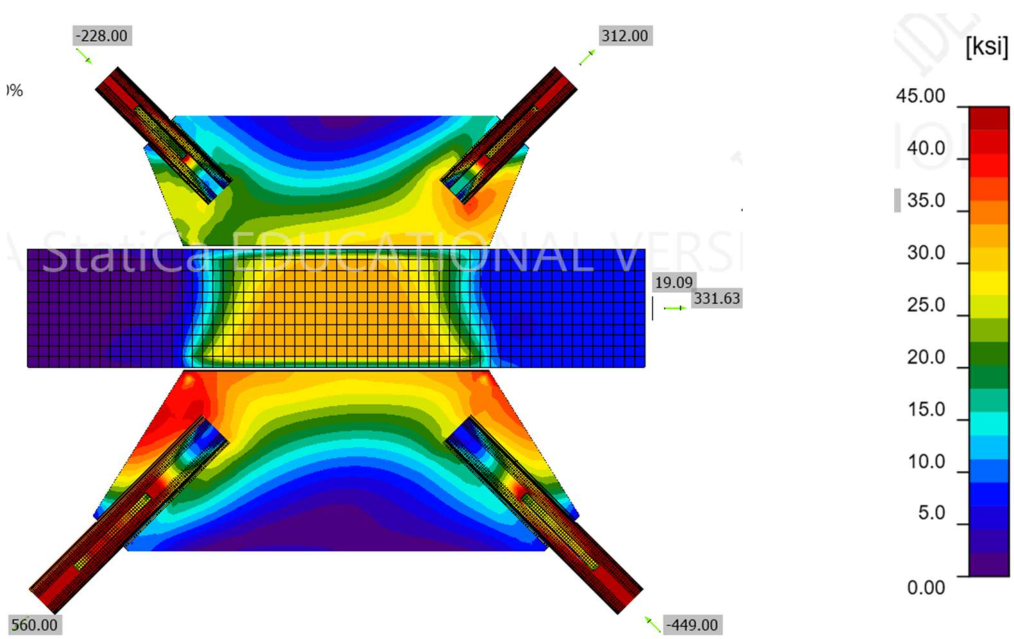
a) Buckling load case   b) Post-Buckling load case

Figure 10 – Required strength of brace connections as per AISC Seismic Provisions Section F2.3(a) and F2.3 (b) - LRFD (AISC 341, 2016)

The "capacity design" analysis of the given connection is performed for the action of design loads in the connecting members in CBFEM. All the four brace members are selected as dissipative items in capacity design. The biggest advantage of IDEA StatiCa Connection is we can add all the load cases, and CBFEM provides us the extreme values amongst the applied loads. The overall check of the given connection for the two design load cases – Buckling and post buckling is verified as shown in Figures 11.



a) Overall check of connection for design loads



b) von Mises stresses in connection for design loads

Figure 11 - Evaluation of connection for design loads - CBFEM

The welded connection between bottom gusset plate and bottom brace subjected to an axial tension design load of 560kips was observed to be failing with the weld utilization of 111.5% >100%, as the weld utilization in CBFEM is based on failure of the critical element. The next step an engineer must take is to either increase the weld size to be 5/16" or to increase the weld length, such that the connection is safe after the analysis of updated connection details.

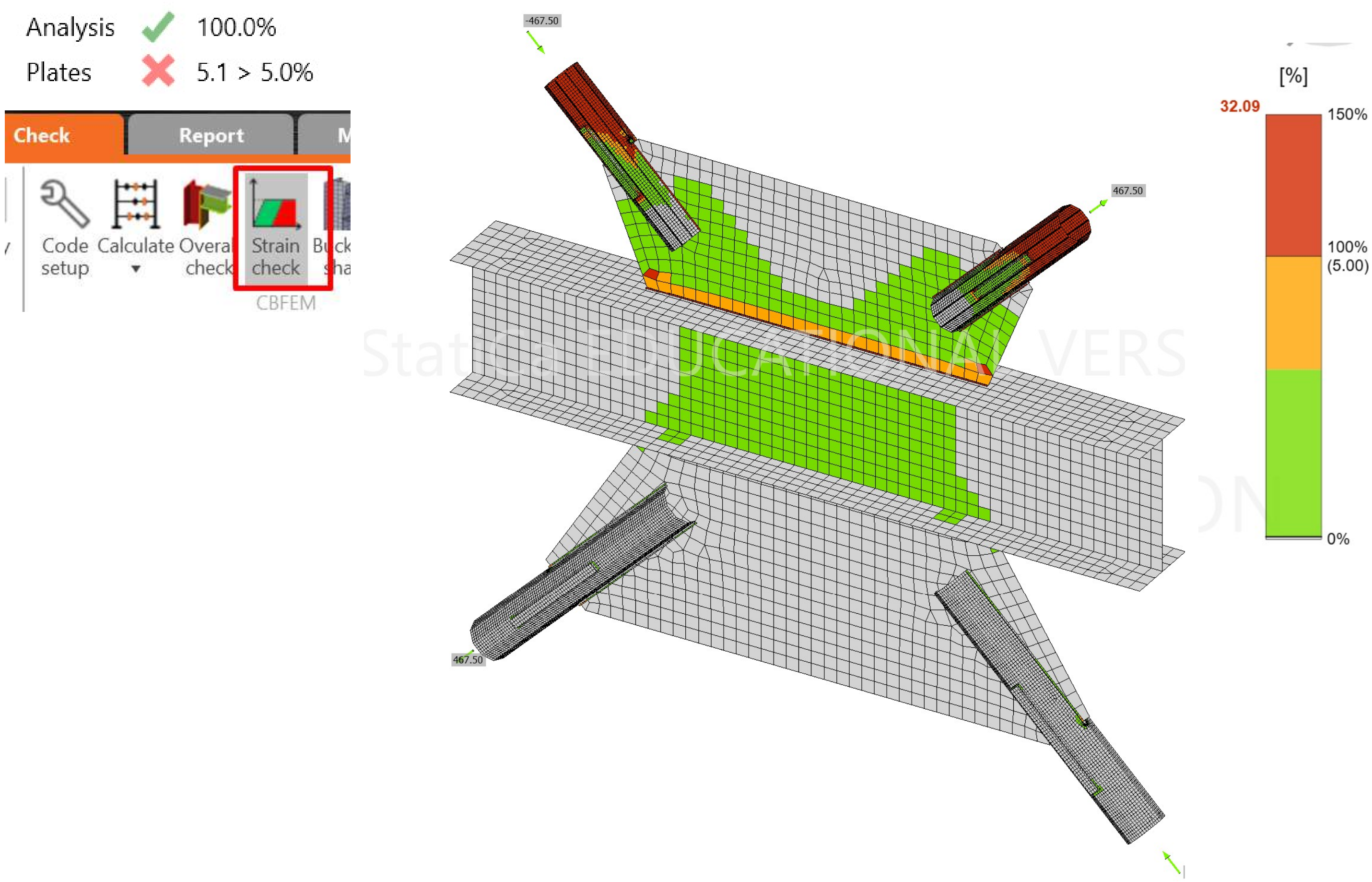
6. Evaluation of limit states for tension load in brace - CBFEM

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.

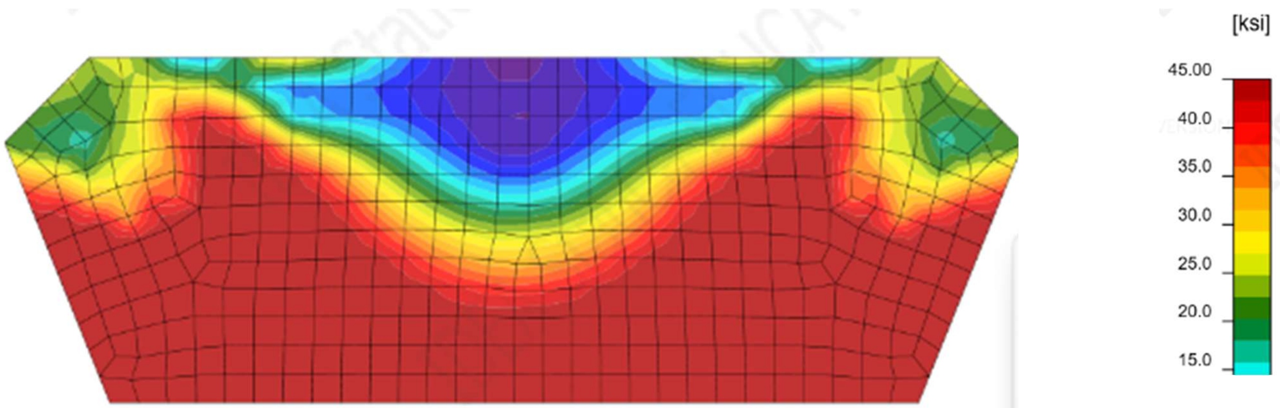
The maximum permitted loads were determined iteratively by adjusting the applied load input to a value that the program deems safe but if increased by a small amount (10kips) until the results would deem unsafe. The focus of this study was to evaluate the limit states related to connection only.

The limit state of tensile yielding for gusset plates was observed in CBFEM for an axial load of more than 450kips (+/-) in all bracing members. When compared to the AISC results, CBFEM yields approximately similar values, but on a higher side with an average difference of 9%. For the gusset plate above beam, the limit state of tensile yielding is observed at an axial load of 467.5kips in brace, as shown in Figure 12 which is 11.2% higher than the capacity as per AISC of 420kips. Similarly, for the gusset plate below beam, the tensile yielding is observed at an axial load of 620kips in CBFEM which is 2.4% higher than the capacity estimated by AISC.

For the limit states of block shear rupture in gusset plates, it was observed in CBFEM for an axial load of more than 500kips (+/-) in all bracing members. When compared to the AISC results, CBFEM yields approximately similar values, but on a higher side. For gusset plate above beam, the limit state of block shear rupture is observed at an axial load of 500kips in brace, as shown in Figure 13 which is 3.25% higher than the capacity as per AISC of 484kips.

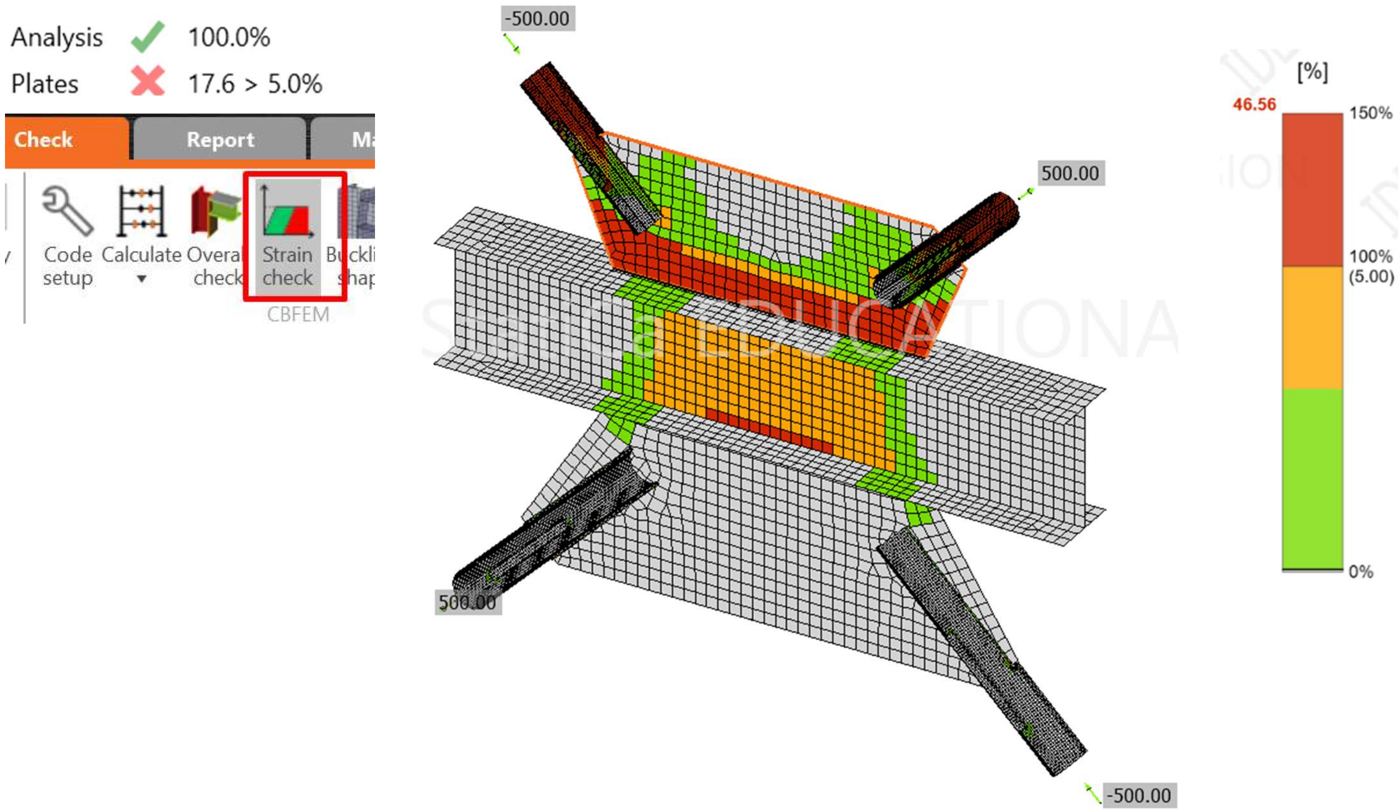


a) Strain check of connection - CBFEM

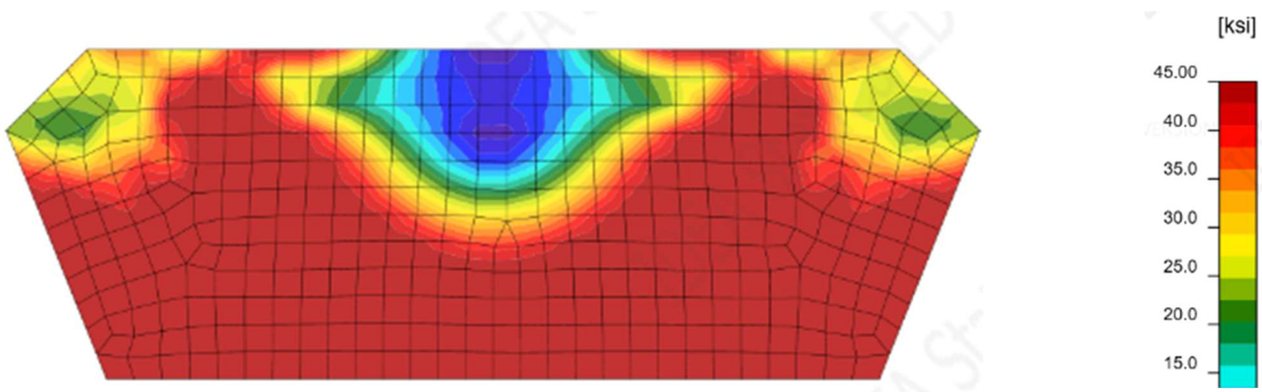


b) von Mises stresses in gusset plate - CBFEM

Figure 12 - Strain check and von Mises stresses in gusset plate for evaluation of limit state of tensile yielding – CBFEM



a) Strain check of connection - CBFEM



c) von Mises stresses in gusset plate - CBFEM

Figure 13 - Strain check, plastic strains and von Mises stresses in gusset plate for evaluation of limit state of block shear rupture - CBFEM

The distribution of forces in a welded connection may be considered either elastic or plastic. An ideal elastic-plastic model is used, and the stress in the weld throat section controls the plasticity state. The check of weld limit state in a connection when done using CBFEM differs from traditional design calculations, which are based on the AISC 360-16. In traditional calculations, small eccentricities, deformations, torsions, Poisson coefficients, etc., may be neglected, which is not the case for the CBFEM wherein all such parameters are taken into consideration, as it is not simply based on a simple 2D statics relation of force distribution and requires to consider the plasticity of the critical element in the analysis and design. The weld model in CBFEM respects the properties of the actual weld, like geometry, stiffness, and ductility. Plastic redistribution is one of the best features CBFEM can offer for welded connections.

A typical example of weld strength computation as per CBFEM is shown in Figure 14; wherein 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.

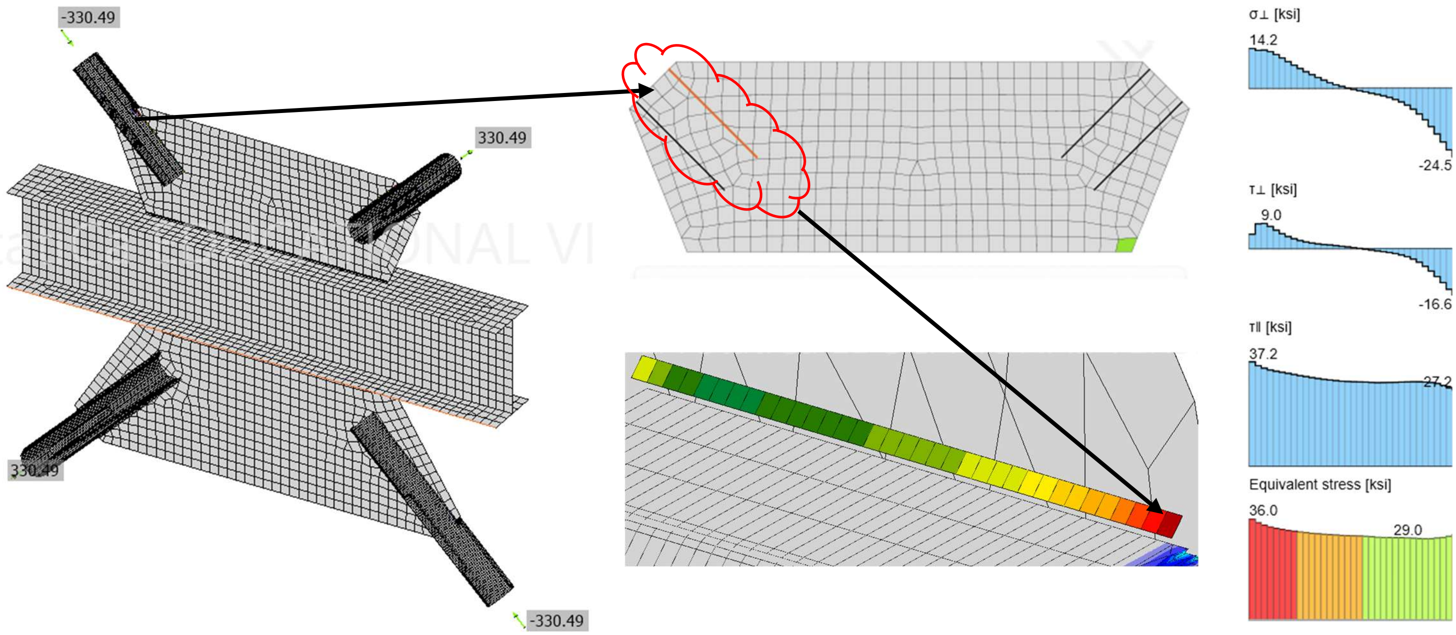


Figure 14: Plastic strains for the evaluation of limit states for fillet weld - CBFEM

The force, F_n, and weld angle, ϕ, are derived from stresses $\sigma_{\perp}$, $\tau_{\perp}$, $\tau_{\parallel}$, length, and effective area of weld finite element. These stresses are the primary output of a finite element solver from IDEA StatiCa. The weld capacity at several locations of the connection obtained as per CBFEM yields similar results when compared with values obtained from AISC, but slightly on a conservative side. As the weld angle obtained from the CBFEM is based on the finite element method, it was usually obtained for the axial loads in braces such that the weld utilization is at least 100%. Overall, CBFEM provides a slightly higher weld size at all locations, when compared with values from AISC as observed from Table 4 and Figures 15 and 16.

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16ths weld size (in)	
			AISC	CBFEM
1	335	Reinforcing plate and brace – Above Beam	5	6
2	370	Reinforcing plate and brace – Below Beam	5	6
3	410	Gusset Plate and Brace – Above Beam	4	5
4	445	Gusset Plate and Brace – Below Beam	4	5
5	546	Gusset Plate to Top Flange of Beam	4	5
6	620	Gusset Plate to Bottom Flange of Beam	7	8

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

Table 4: Comparison of fillet weld size as per AISC and CBFEM

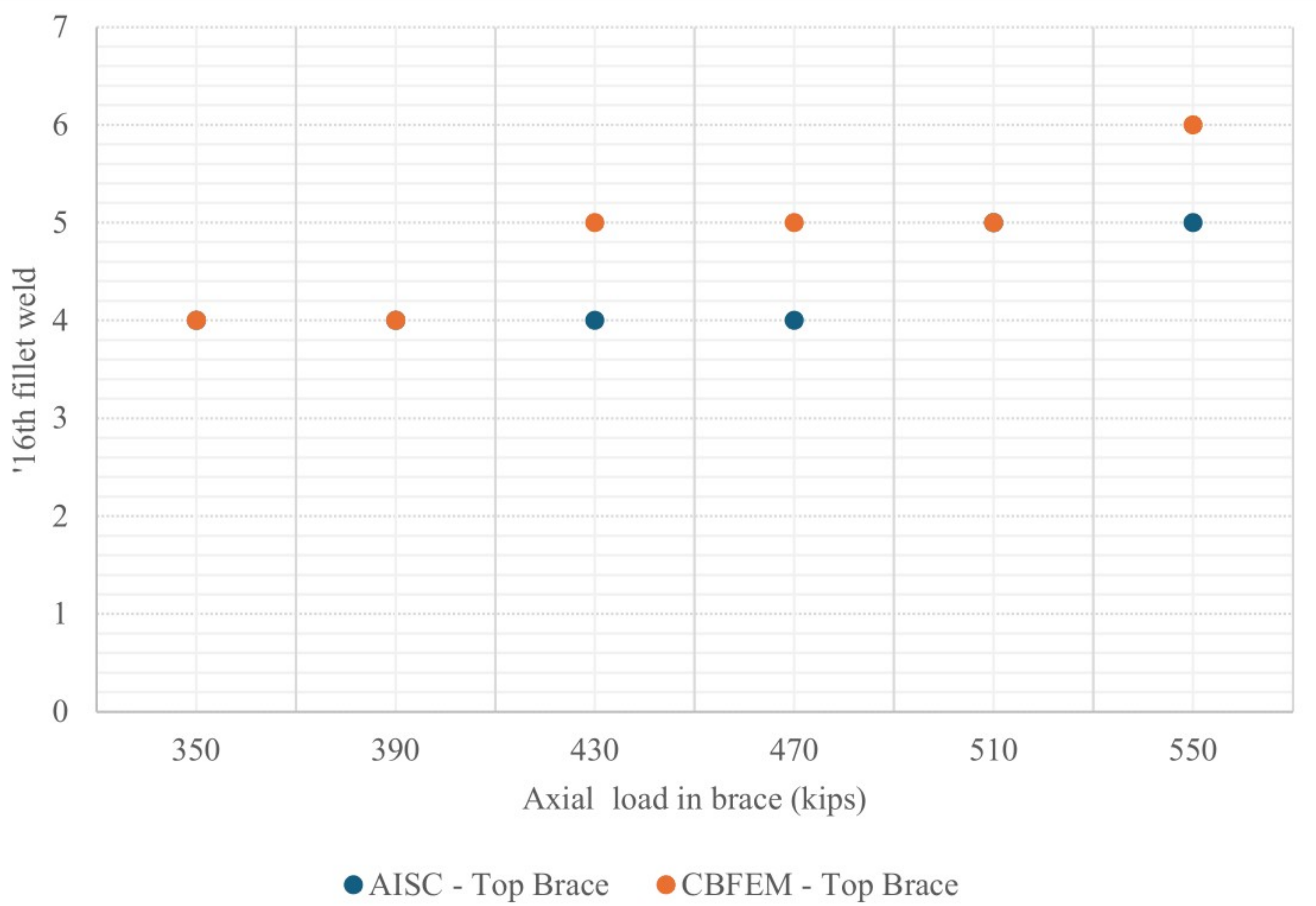


Figure 15: Comparison of required size of fillet weld at brace-to-gusset plate above beam

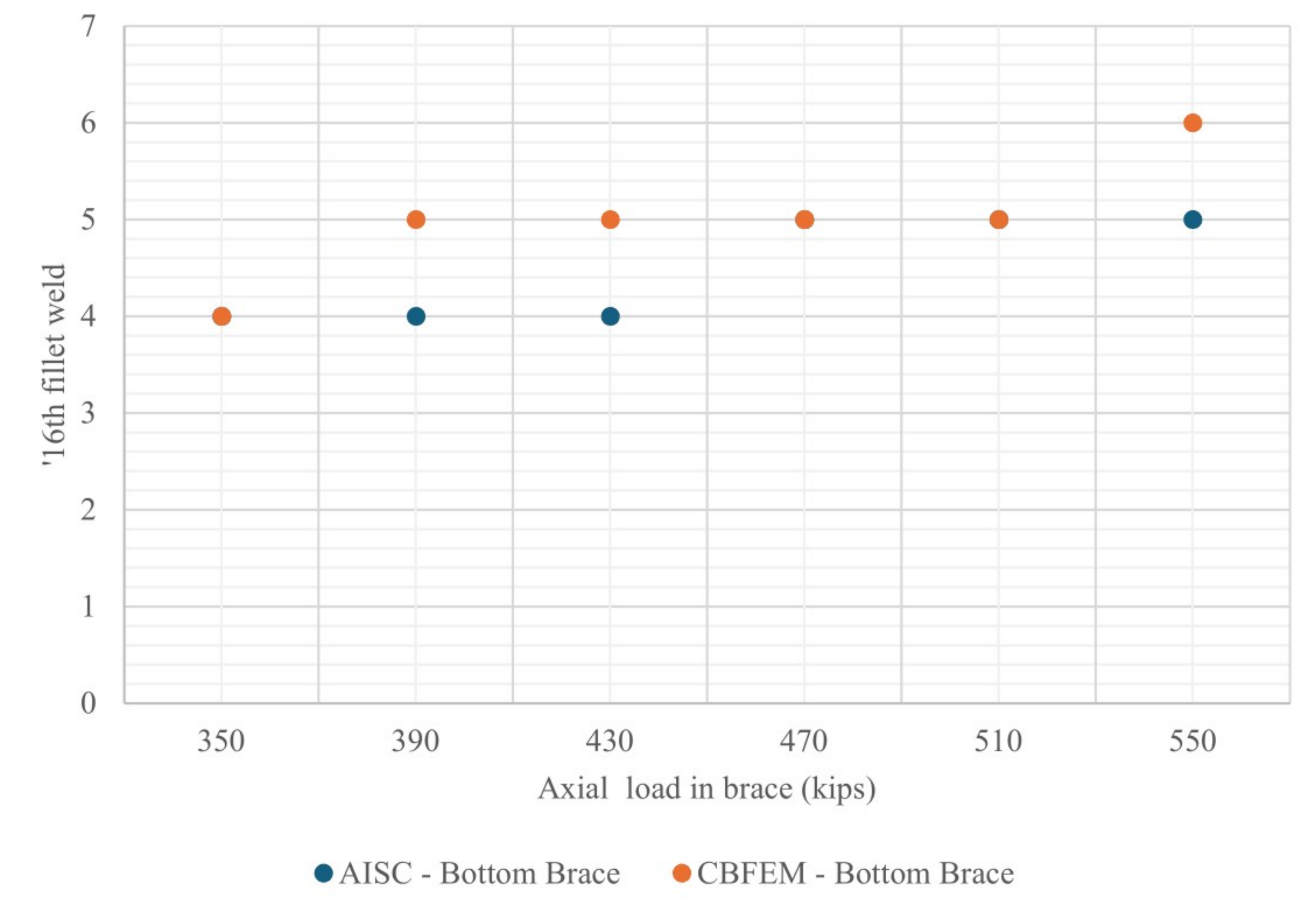
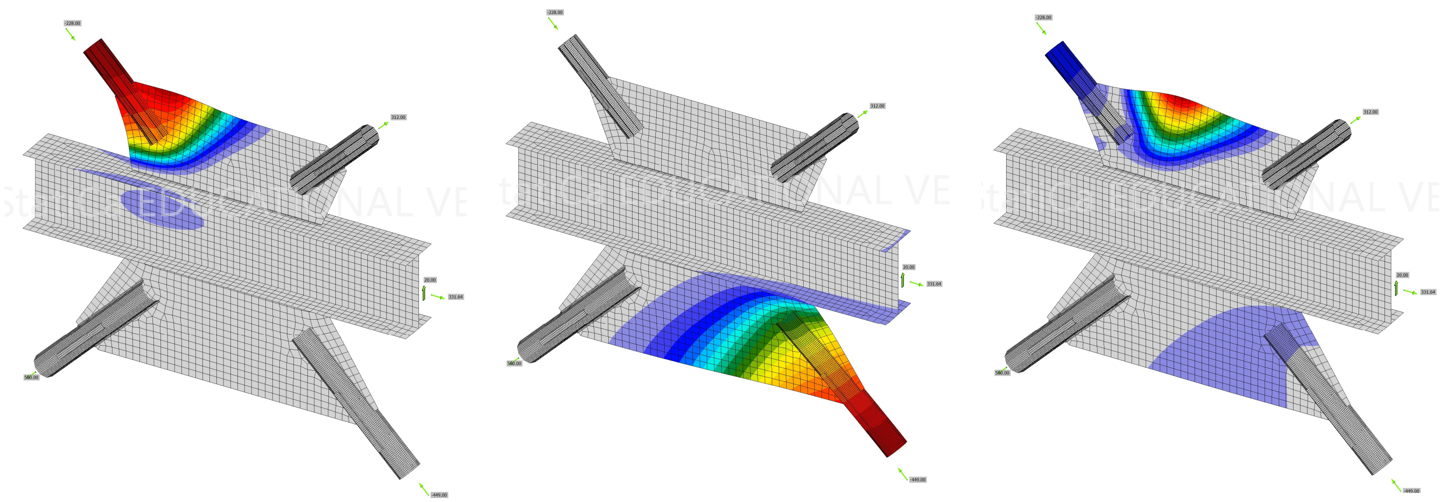


Figure 16: Comparison of required size of fillet weld at brace-to-gusset plate below beam

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

As brace members in a special concentrically braced frame (SCBF) are designed to resist the high cyclic loading of an earthquake, a brace will buckle for an action of large compression load, which results in a significant loss of the brace strength and the connecting gusset plate.

Using the buckling multiplier factor and mode shapes obtained in CBFEM, we can check the buckling capacity requirement as per AISC for the gusset plate. Currently, it is the only measure, and therefore, it is hard to differentiate between the buckling resistance of various connecting parts, e.g., buckling of the gusset plate on the Whitmore section or gusset plate sideways buckling in CBFEM.



a. Buckling mode shape 1 Buckling factor as per CBFEM = 3 (Deformed shape factor = 10)	b. Buckling mode shape 2 Buckling factor as per CBFEM = 3 (Deformed shape factor = 10)	c. Buckling mode shape 3 Buckling factor as per CBFEM = 5 (Deformed shape factor = 10)

Figure 17: First three buckling mode shapes for the X-brace chevron connection - CBFEM

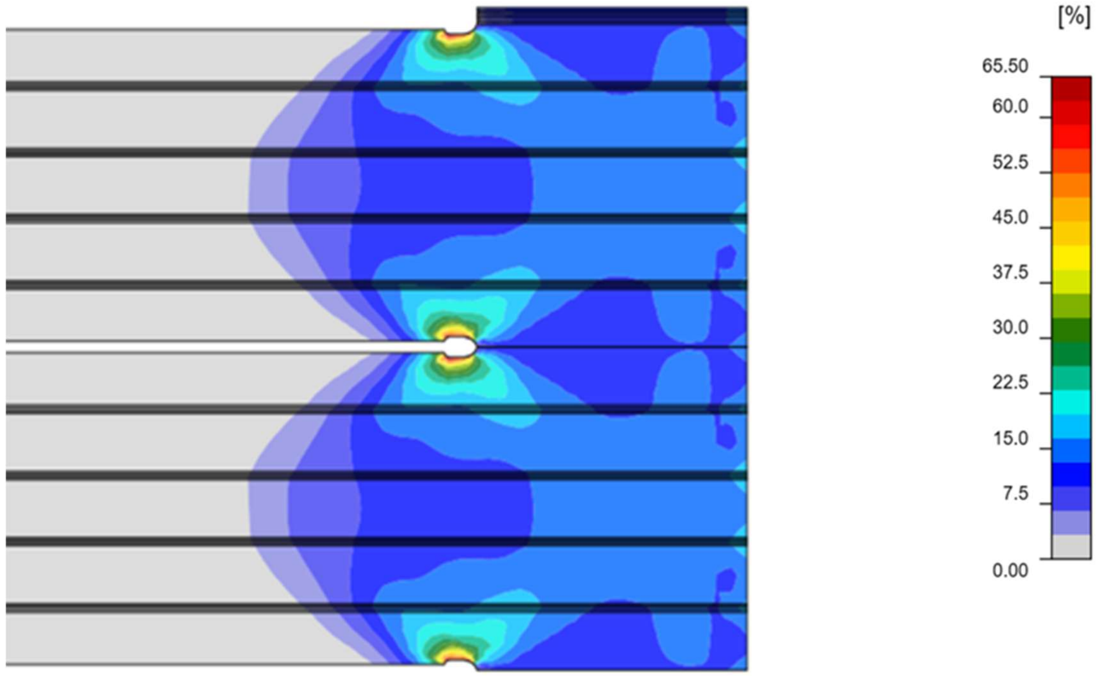
In mode shape 1, buckling of gusset plate above the beam is observed to be governing, Whereas, in mode shape 2, the buckling of bottom brace subjected to the axial compression load of 449kips is observed. For third mode, the mode shape depicts the buckling to occur in the gusset plate above beam.

As per the results from CBFEM shown in Figure 17, the buckling factor for critical mode (mode1) is 3 for an axial compressive load of 228 kips in brace. Even though the entire gusset plate is connected to only one side, the plate is restrained in two sides. One is the connection to the beam and the second is the restriction from the other brace which is in tension. Hence, it is concluded that the category of the buckling issue for this connection is local buckling. For local buckling category, since the obtained buckling factor of 3 for critical mode is less than the required buckling factor of 3 for member plate, the given connection is safe for the buckling limit states in both AISC as well as CBFEM.

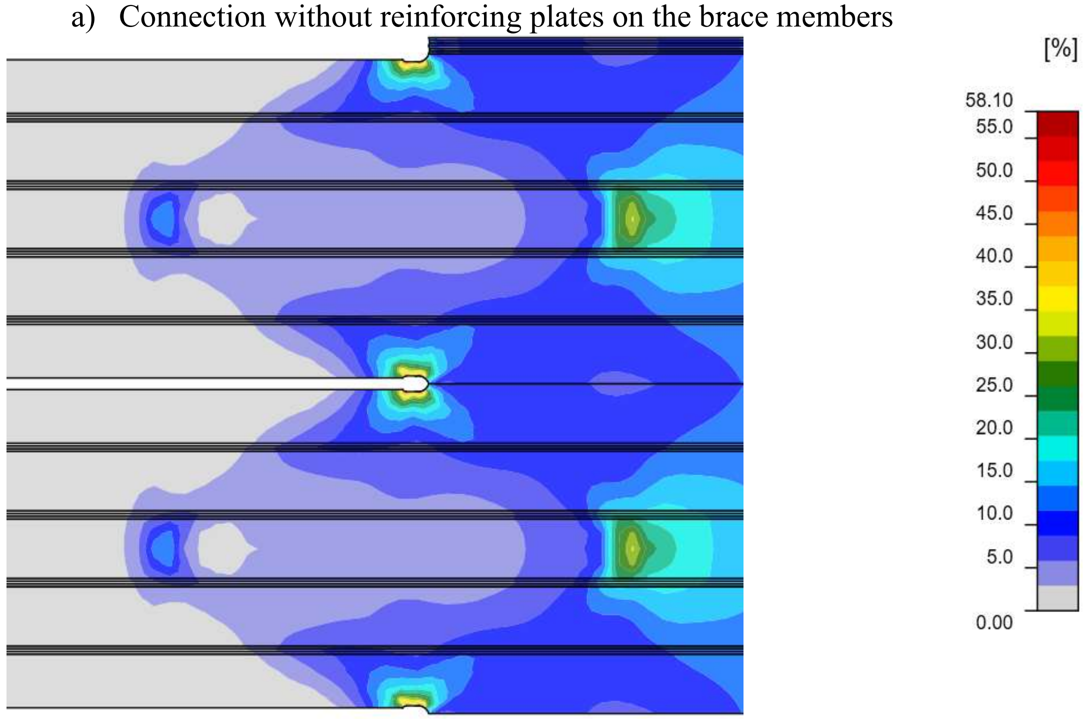
8. Parametric Studies

a. Influence of reinforcing plates on brace members

It is a very common practice to weld a slotted HSS brace to gusset plate, which tends to reduce the effective net area of the brace. To compensate for the loss of this effective net area in braces, it is essential to provide reinforcing plates for the HSS brace members. The influence of these reinforcing plates is evaluated in this parametric study for an action of 400 kips of axial tension load in brace.



a) Connection without reinforcing plates on the brace members



b) Connection with reinforcing plates on the brace members

Figure 18 – Plastic strains in connection for evaluation of influence of reinforcing plates on the brace members

Based on the observations from the Figure 16 , 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.

Also, the connection without the reinforcing plates on brace are observed to have higher overall weld utilization (lower weld capacity) with an average difference of around 15% as shown in Figure 17.

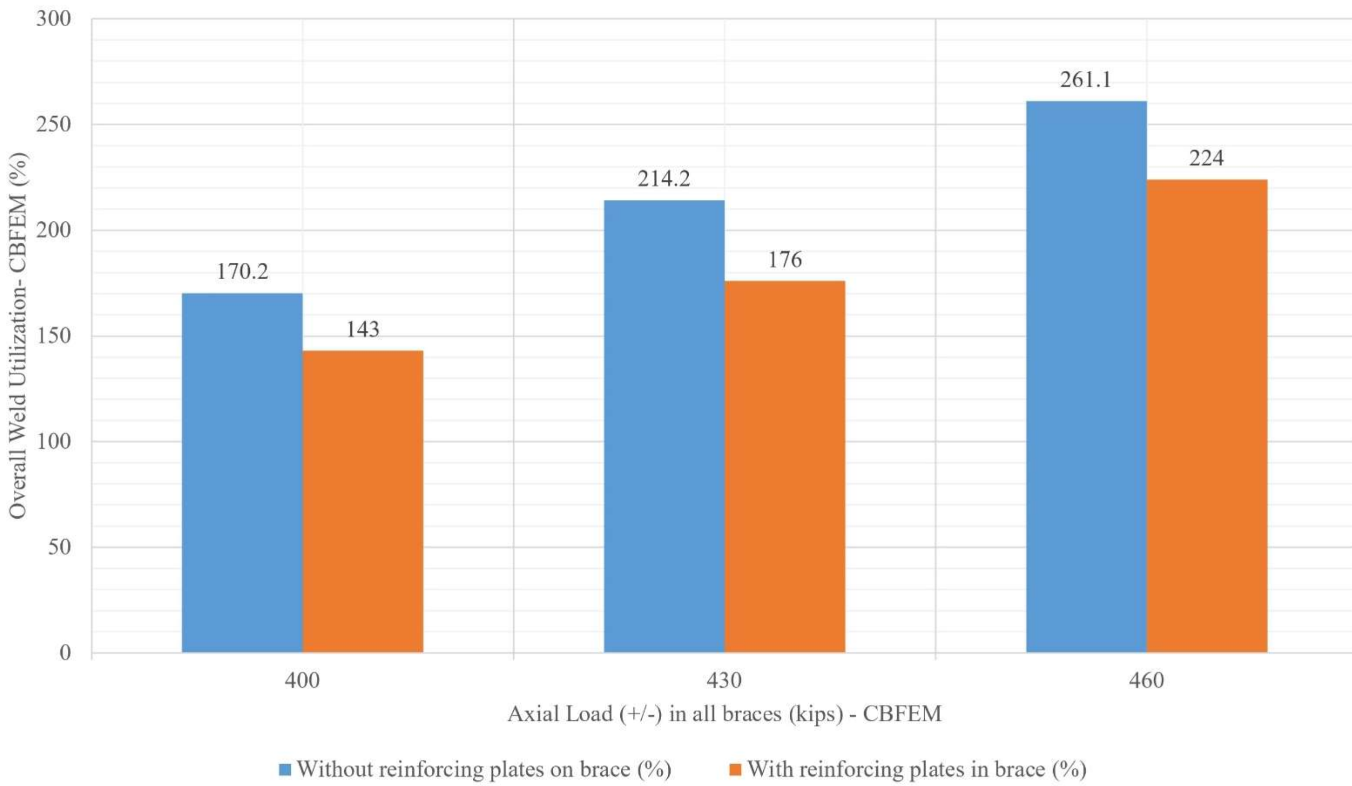


Figure 19 – Evaluation of influence on overall weld utilization of connection - CBFEM

b. Changing fillet welds at brace-to-gusset location to CJP welds

The failure of fillet welds at brace-to-gusset locations was observed as one of the governing limit states for the action of design loads. Therefore, these fillet welds are changed in the design model to full-penetration butt welds (CJP welds). CJP groove welds and butt welds are modeled by directly connecting the components using multi-point constraints which does not introduce any flexibility in IDEA StatiCa. As the strength of CJP groove weld is controlled by base metal, their strength in CBFEM is not checked.

Based on this modification, the load at braces can be increased up to 392 kips before a limit state of weld is observed at beam-to-top gusset plate from results as shown in Figure 18. Overall, there is a 19% increase in the load carrying capacity of the given connection for the performed modification.

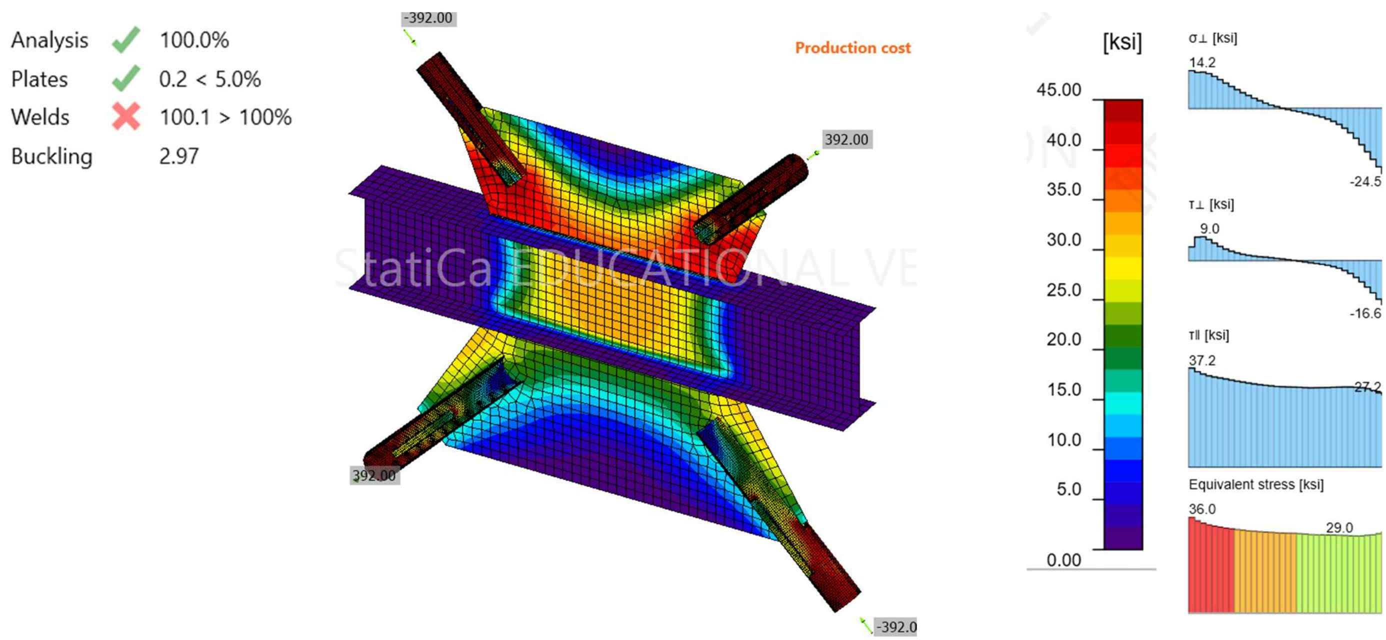


Figure 20: von Mises stresses in connection for change of fillet welds to CJP welds at all brace-to-gusset locations

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 5 and the plot of results is shown in Figure 19 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	198	228	1.5	85	Gusset Plate	Not OK
2	1/2	HSS6x0.312	324	228	3	171	Gusset Plate	OK
3	¾	HSS6x0.312	563	228	3	171	Brace member	OK
4	1	HSS6x0.312	790	228	3.6	205.2	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 $P_{b\_CBFEM} = (P_{comp} x\ \alpha_{cr})/4$.

Table 5 – Parametric Study for buckling of top gusset plate for multi-story X-brace connection

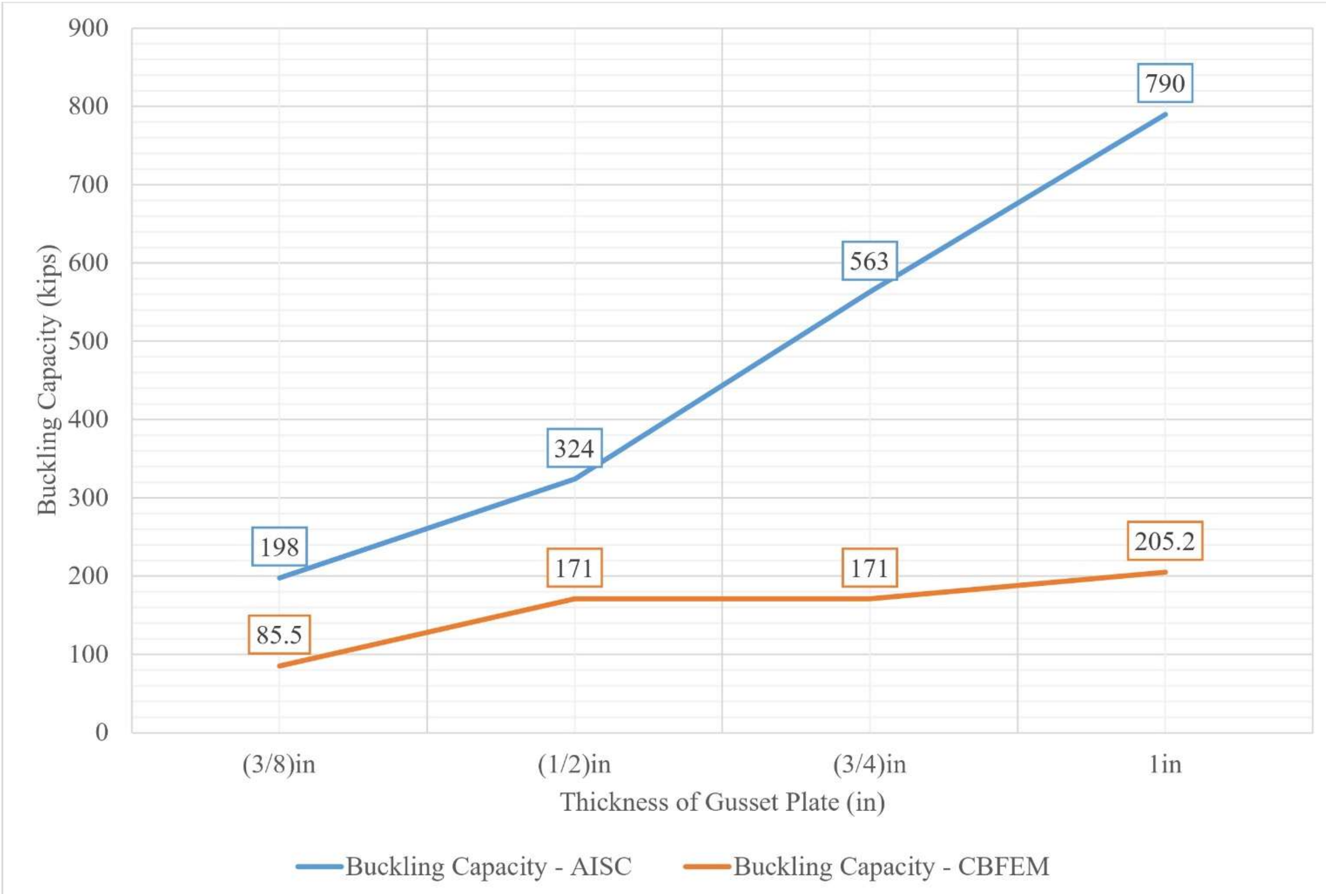
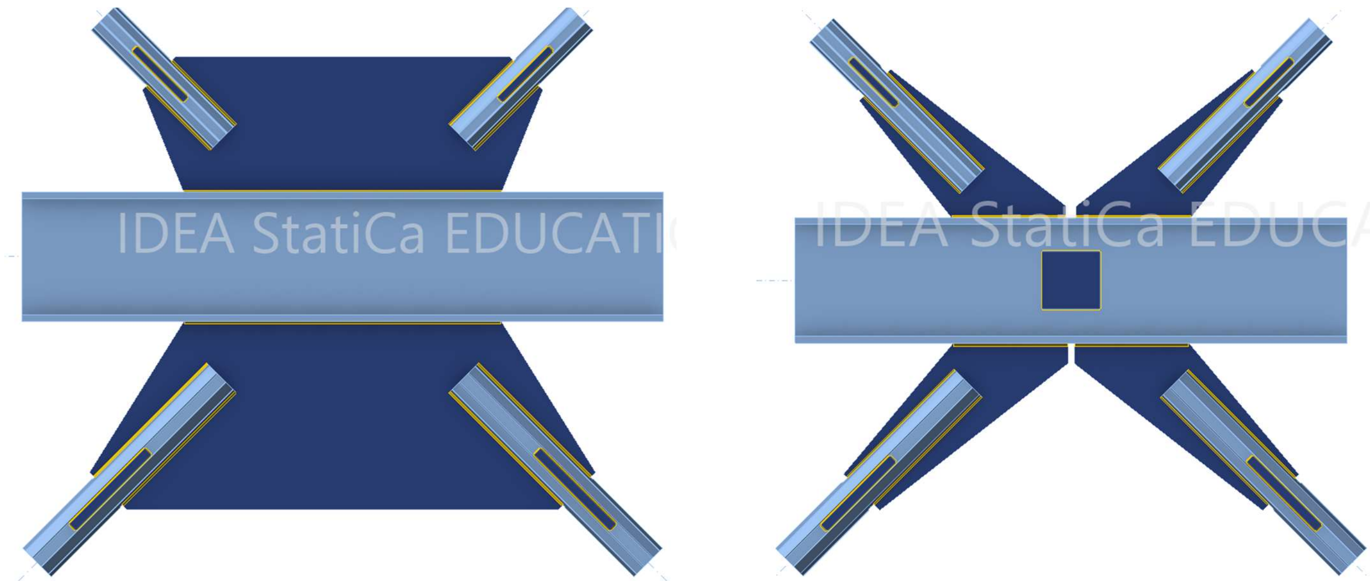


Figure 21 – Curve of Buckling capacity vs thickness of gusset plate

d. Comparison of alternate connection design

An alternate connection design consisting of individual gusset plates for each brace member as given in example 5.3.8 of AISC Seismic Design manual was considered in CBFEM for this parametric study as shown in Figure 20.



a) Example 5.3.7, AISC 341-16 b) Example 5.3.8 , AISC 341-16

Figure 22: Comparison of alternate connection design

As the limit state of weld connection was governing one, the same was evaluated in detail. Overall, the alternate design was found to have slightly higher weld utilization (lesser weld capacity), with an average difference of around 7% in results with the connection having single gusset plate in each side as shown in Figure 21.

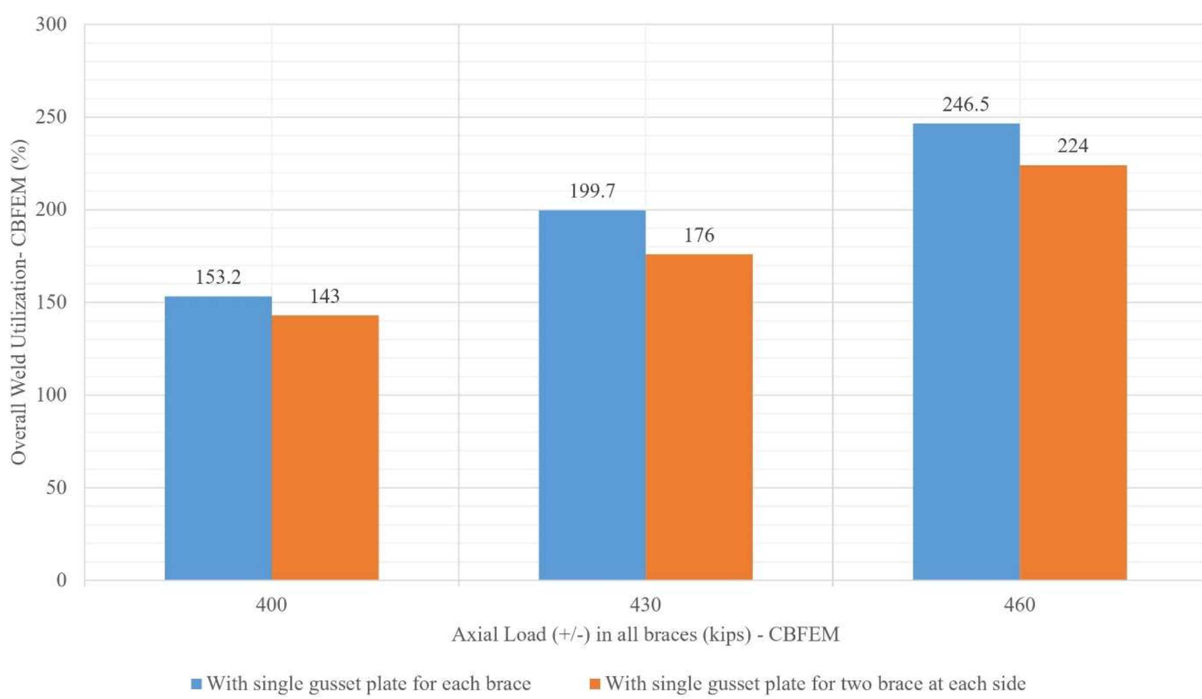


Figure 23: Comparison of overall weld utilization for alternate design

Next, for the case of plates and member, maximum strains are found to be concentrated around the beam web, for the case of higher axial loads in the brace. Overall, the difference in results for various limit states for plates and members was observed to be within 5% when compared with the connection with the single gusset plate on each side of the brace. The plastic strains in the beam web for the axial load (tension or compression) of 400kips in brace is shown in Figure 22.

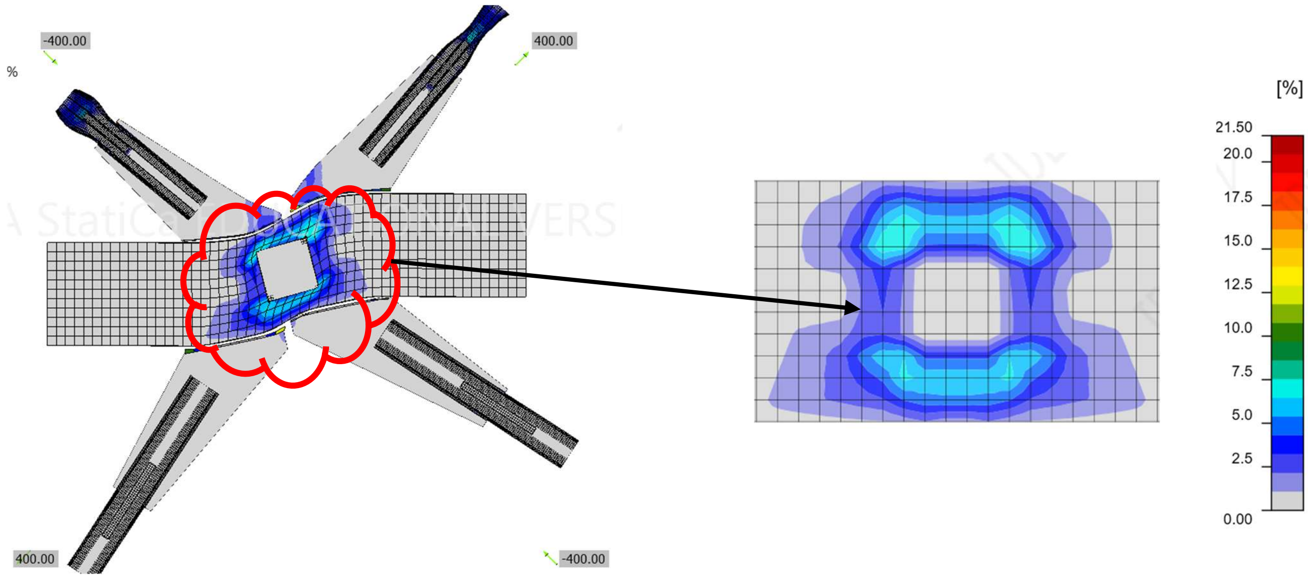
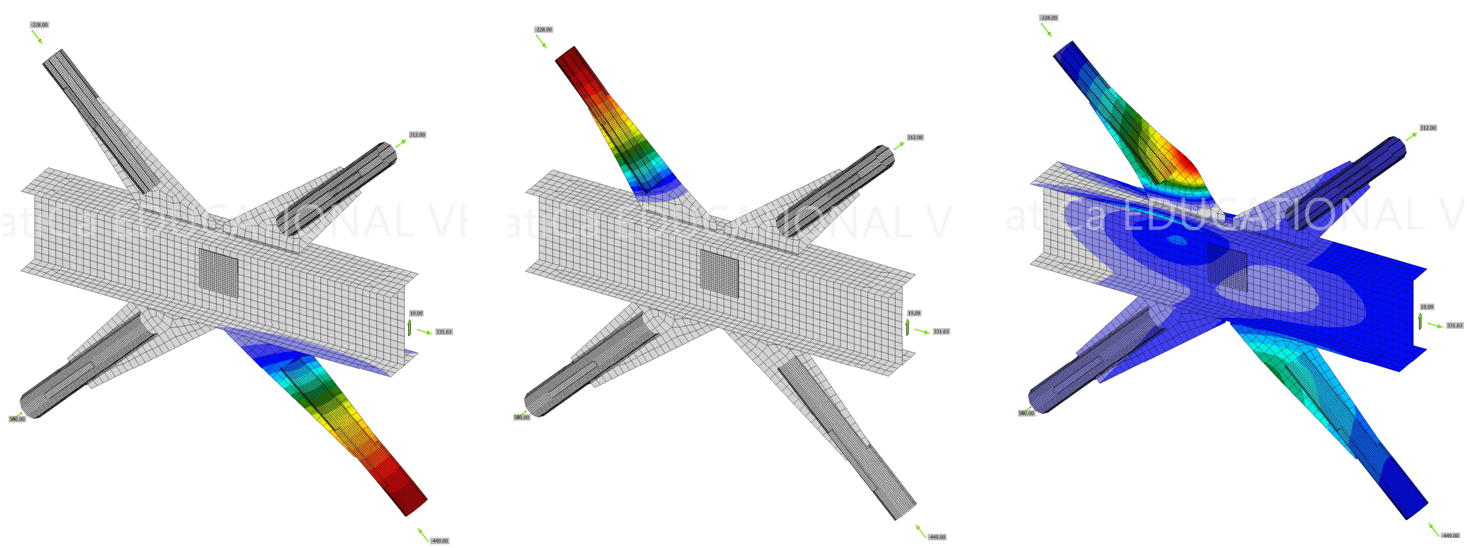


Figure 24: Plastic strains near beam web for higher axial loads

For the case of buckling, the first three modes are evaluated in detail. For the critical buckling mode shape 1, the elastic buckling factor was found to be 9.9 and, in the brace, subjected to an axial compression load of 228kips above the beam as shown in Figure 23.



a) Buckling mode shape 1
Buckling Factor as per CBFEM = 4
(Deformed shape factor = 10)

b) Buckling mode shape 2
Buckling Factor as per CBFEM = 4.35
(Deformed shape factor = 10)

c) Buckling mode shape 3
Buckling Factor as per CBFEM = 9.85
(Deformed shape factor = 10)

Figure 25: First three buckling mode shapes for the alternate design

For the given connection, as the gusset plates are restrained only at one side towards the beam via weld connection, the buckling issue corresponds to global buckling. Since the critical buckling factor is less than 9 and observed in the brace member, this requires further investigation in IDEA StatiCa Member by performing non-linear buckling analysis.

It can be concluded that the actual design consisting of individual gusset plates at each side is the effective design for the action of higher axial load.

9. Flowchart

High quality or resolution of flowcharts as pdf in "Flowchart" folder of each example.

10. Comparison of values

Sr No	Limit State of	AISC (kips)	CBFEM (kips)	Δ_{AISC_CBFEM} (%)
1	Tensile Yielding of Gusset Plate – Above Beam	420	467.5	11.2
2	Block Shear Rupture of Gusset Plate – Above Beam	484	500	3.2
3	Tensile Yielding of Gusset Plate – Below Beam	605	620	2.4

Sr No.	**P_{brace} (kips)**	**Check of weld connecting**	**Required '16^{ths} weld size (in)**	
			AISC	**CBFEM**
1	335	Reinforcing plate and brace – Above Beam	5	6
2	370	Reinforcing plate and brace – Below Beam	5	6
3	410	Gusset Plate and Brace – Above Beam	4	5
4	445	Gusset Plate and Brace – Below Beam	4	5
5	546	Gusset Plate to Top Flange of Beam	4	5
6	620	Gusset Plate to Bottom Flange of Beam	7	8

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

11. Conclusion

1. CBFEM can predict the actual behavior and failure modes for the presented connection.
2. Difference between the equivalent plastic strain and maximal equivalent plastic strain was observed as shown in Figure 9 for the case of capacity design, which was approximately the same in case of stress-strain analysis. It requires further investigation.
3. Weld size as computed by CBFEM is slightly on the higher side when compared to 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, thereby providing a higher weld angle, and hence a higher weld capacity for some cases.
4. 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.
5. For bracing connections in SCBF, reinforcing plates on the brace members allows the inelastic demands to develop along the brace and away from the connection region. Thereby, providing a higher capacity for brace.

12. Suggestions for Improvement

a. Joint Design Resistance for seismic vertical bracing connections.

b. “Loads in equilibrium” not an option in capacity design as of V23.1.

13. References

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