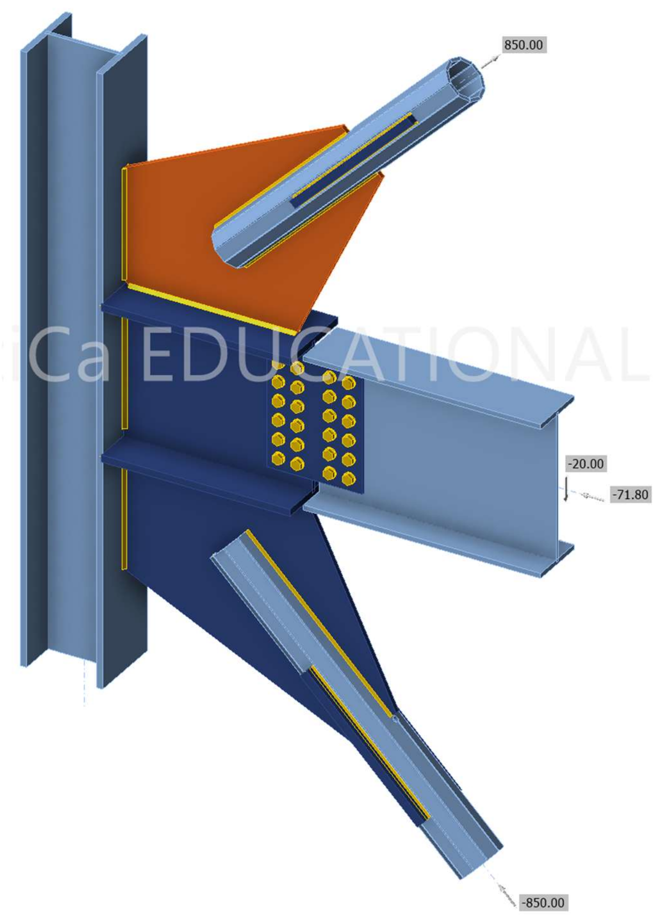
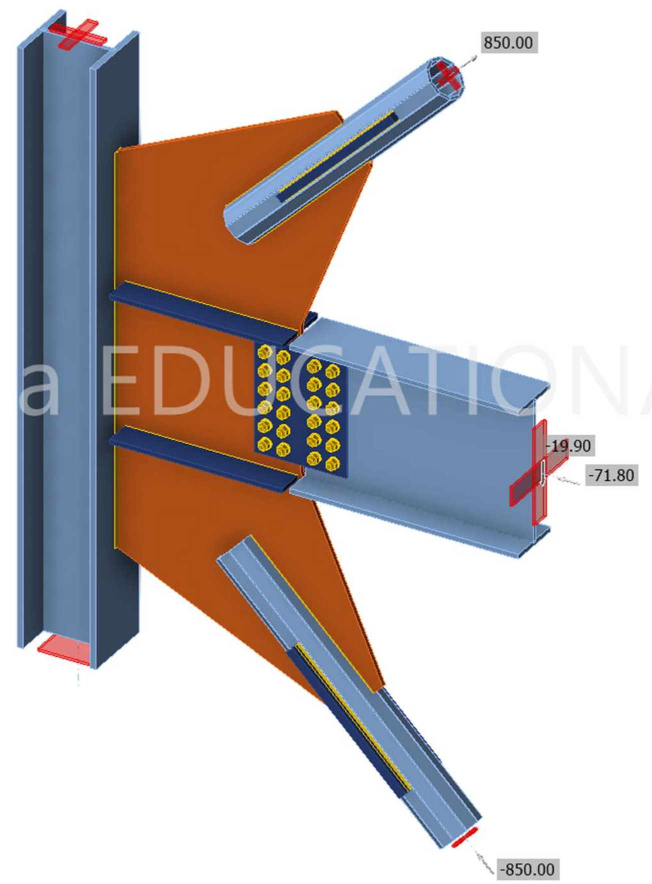
Corner Brace-to-Beam / Column Connection in a Two-Story X-Brace Configuration of Special Concentrically Braced Frame (SCBF) System





This is the fourth verification example in CBFEM from a series of seismic vertical diagonally braced connections wherein a corner brace-to-beam / column 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 V23.1.

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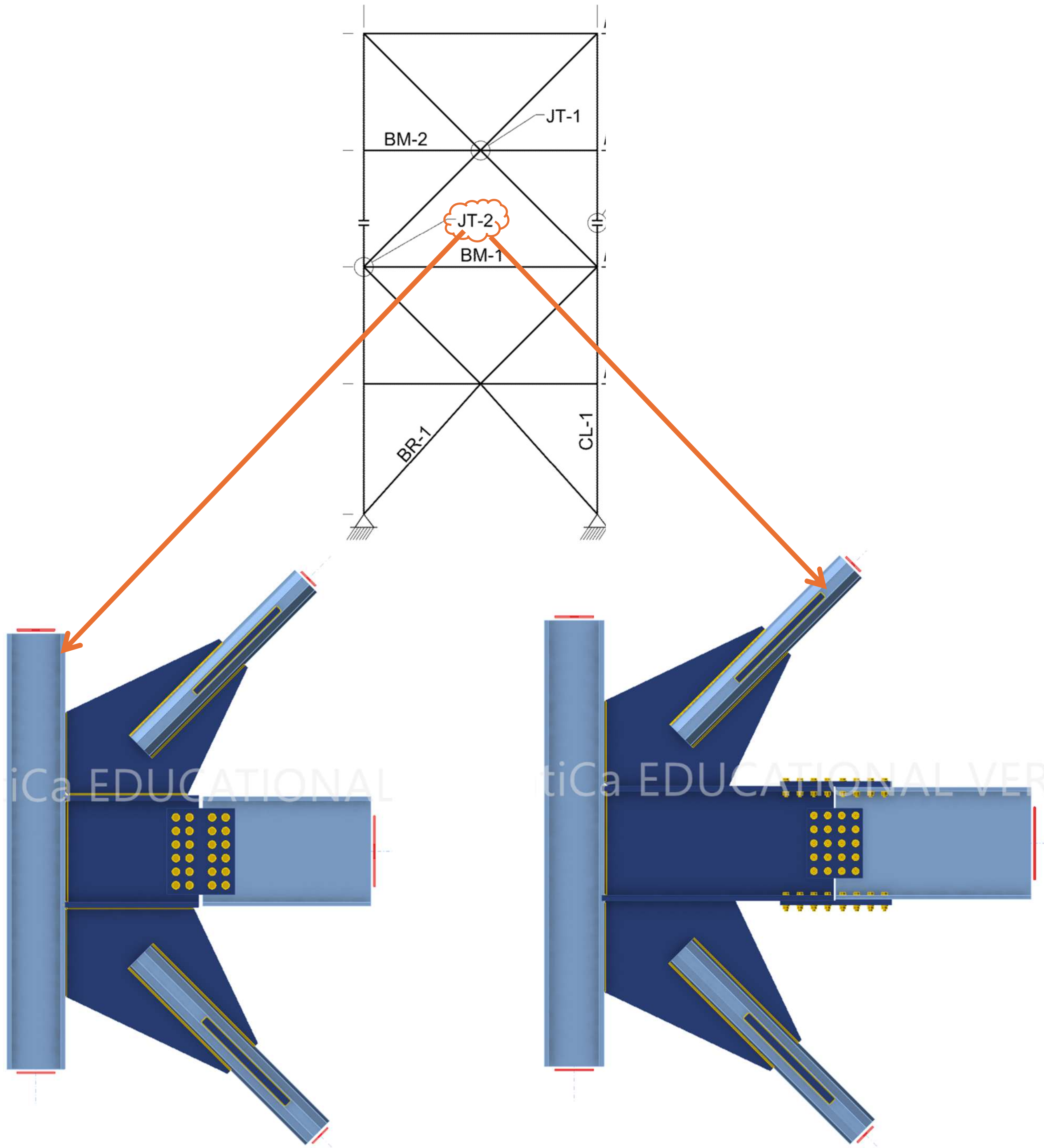
1. Introduction



a) Simple / Shear connection at beam-to-stub beam   b) Moment connection at beam-to-stub beam

Figure 1 – Types of brace-to-beam/column corner connection in SCBF

The brace-to-beam/column corner connection must be designed as a simple / shear connection or as a moment connection in a special concentrically braced frame (SCBF). For simple connection, a moment release shear splice plate (1-NS and 1-FS) is provided away from connection region, which is connected to beam and stub web by bolts as shown in Figure 1.a). As per section F2.6b.(a) of AISC341-16, the connection assembly when designed as a simple connection shall meet the requirements of Specification Section B3.4a, wherein the required rotation is accounted to be 0.025 radians. The use of such simple connection usually allows the beam to rotate relative to the column, which is very essential, as the brace frames are expected to have large story drifts, which thereby results in development of large rotations between beam and column. The rotation for such a joint can lead to poor frame performance, if it is not accounted for in the connection design.

Alternatively, the beam-to-column connection may be designed to resist a moment equal to the beam flexural strength, $(1.1R_yM_p)/\alpha_s$, or a moment equal to the sum of expected column flexural strengths, $\sum(1.1R_yF_yZ)/\alpha_s$ as given in section F2.6b.(b). In this case, a fully restrained moment connection consisting of flange plates and moment release shear splice plates, along with enough filler plates in beam away from connection region must be provided as shown in Figure 1.b), meeting the same requirements as ordinary moment frames, as required. While this could increase system strength significantly, it will also add up to the overall connection cost of the braced frame.

2. Problem Description

The objective of this example is to verify the component-based finite element method (CBFEM) for a corner brace-to-beam / column connection in two story X-braced configuration of a special concentric braced frame (SCBF) of steel building. The brace is designed to buckle out-of-plane, for a linear hinge zone or fold line of thickness "2t" in gusset plate, such that t is the thickness of the gusset plate.

The floor plan details of considered building, along with the connection details as provided in example 5.3.9 for SCBF in AISC341-16 are considered as presented in Figure 2 and 3 respectively. The results obtained from calculation method 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 Connection version 23.1.

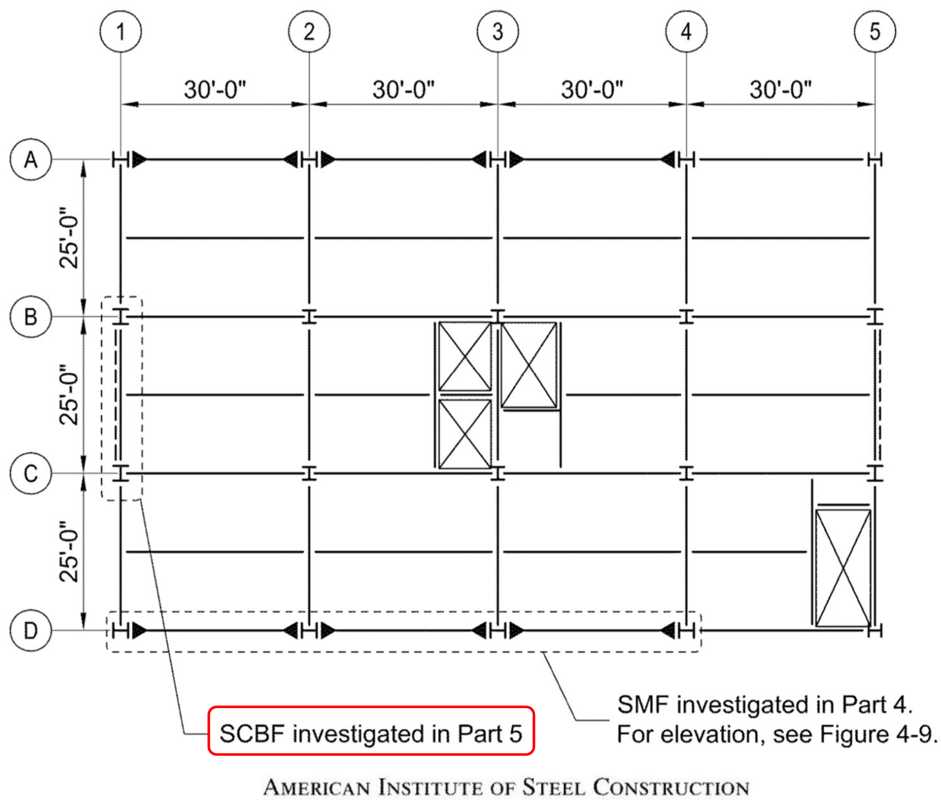


Figure 2 - Floor plan of steel building highlighting the investigated SCBF (AISC 341, 2016)

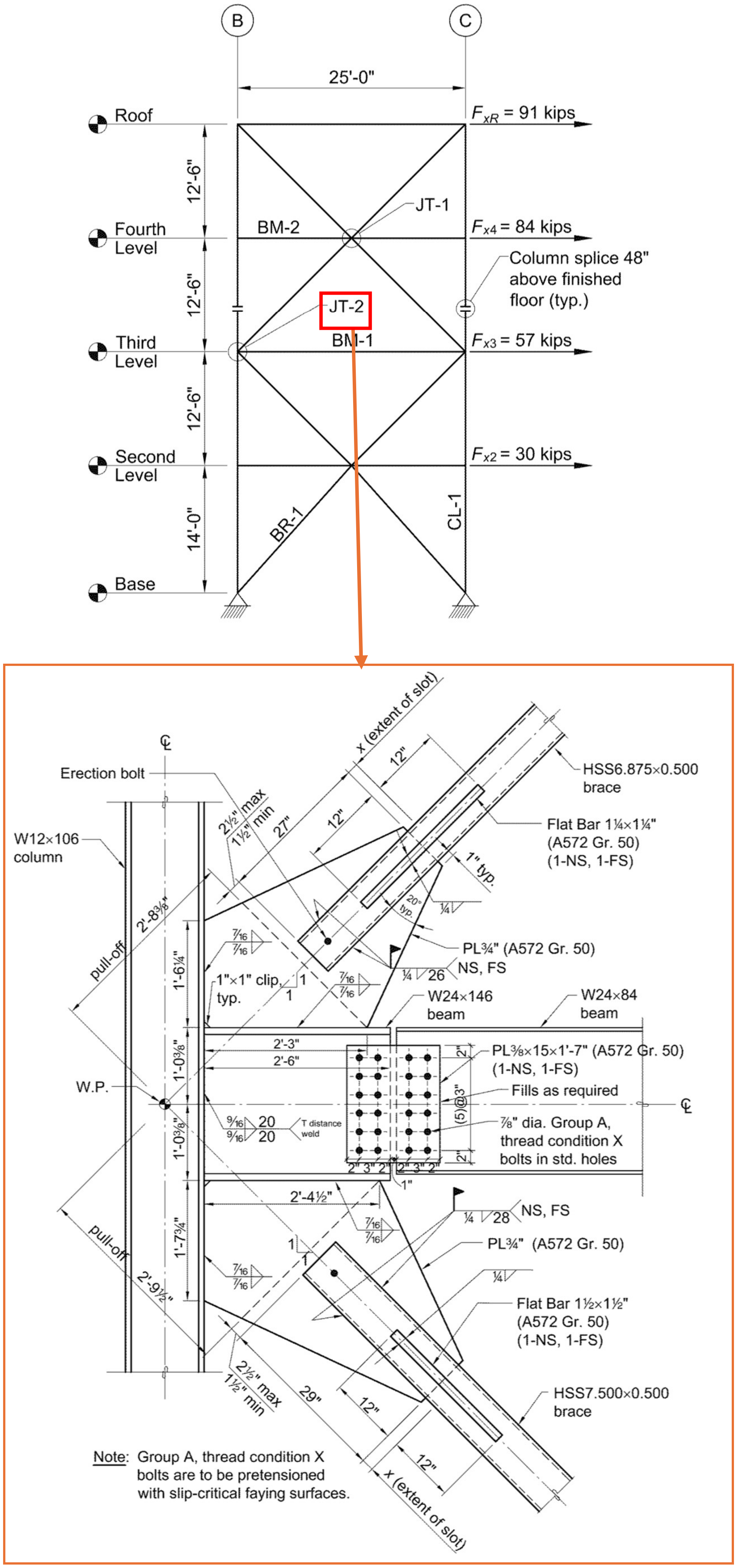


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Figure 3 - Braced frame elevation and considered corner connection details (AISC 341, 2016)

In most design practices, a beam is considered continuous from column-to-column. For the presented connection design example, a splice is inserted at 2'-6 ½" from the face of the column. This splice must carry the loads that would exist at this point in the continuous beam and are designed as extended shear plate as presented in AISC *Manual* Part 10, along with the check for splice ductility such that $2t < t_{max}$, where t is the gusset plate thickness and t_{max} is the maximum splice plate thickness permitted and calculated using Equation 10-3 from AISC 360-16.

This connection design example considers the use of a beam shear splice plate away from the connection region, and connects the beam stub and link beam section, such that the link beam section has a smaller cross-section than the beam stub and it provides a simple beam-to-column connection satisfying the Section F2.6b(a) of AISC341-16. The purpose of providing a larger section for the beam stub is to effectively meet the higher shear demands arising from the braces over the connection due to seismic forces. The applicable building code specifies using ASCE/SEI 7 for calculating the loads.

The details for the member as well as connection are as presented below,

Member Details

1. **Column cross-section**

- W12x106, ASTM A992

2. **Stub Beam cross-section**

- W24x146, ASTM A992

3. **Link Beam cross-section**

- W24x84, ASTM A992

4. **Brace cross-section**

- Top brace as HSS6.875x0.5
- Bottom brace as HSS7.5x0.5
- ASTM A500, Grade C Round

Plate Details

1. **Gusset Plate**

- Both gusset plates as ¾", ASTM A572-50

2. **Reinforcing Plate on HSS Brace**

- PL 1 1/4"x1 1/4" (1-NS and 1-FS) on brace above beam, ASTM A572-50

Connection Details

1. **Bolted Connection (Only at shear splice plate)**

- Group A Bearing Type Bolts with threads excluded from the shear plane (thread condition X),
- 7/8" diameter bolts; A325-X

2. **Welded Connection**

- E70xx electrode
- 7/16" double sided fillet weld connecting gusset plate to flanges of beam and connecting gusset plate to column flanges.
- 1/4" rear sided fillet weld connecting brace and gusset plates.
- 1/4" double sided fillet welds connecting all stiffening plates and HSS brace wall.
- 9/16" double sided fillet welds connecting beam stub web-to-column.
- CJP groove welds at the beam flanges-to-column (Demand critical welds)

- PL 1 1/2”x1 1/2” (1-NS and 1-FS) on brace below beam, ASTM A572-50

3. Beam Splice Plate

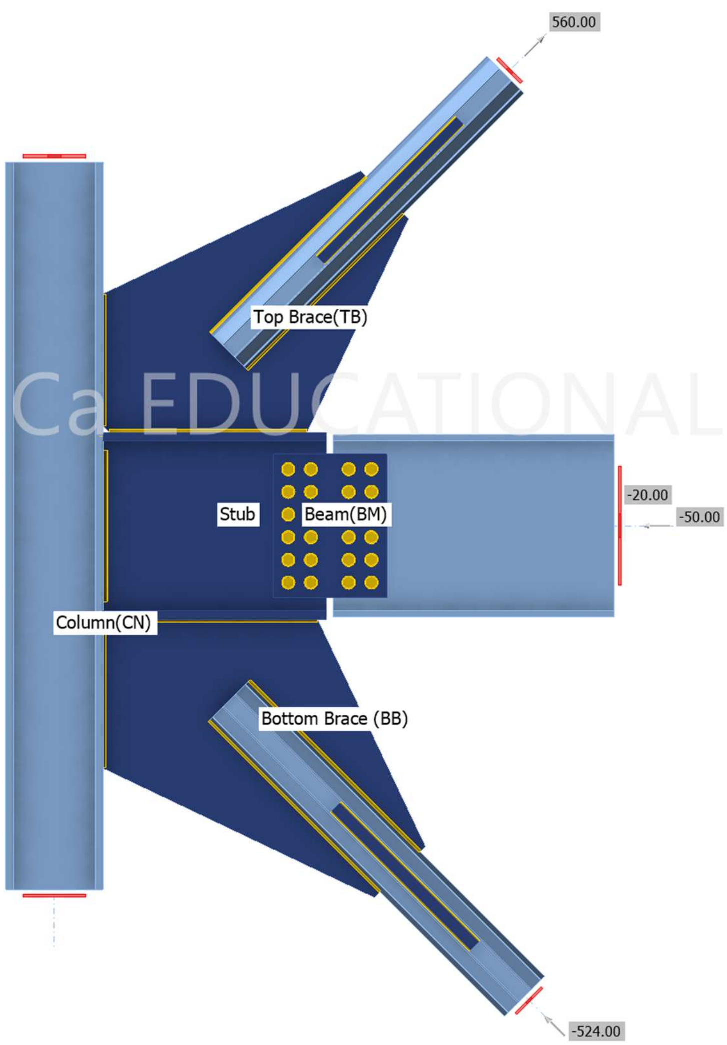
- Two PL (1-NS and 1-FS) 3/8”x15x1’-7” connecting web of stub beam and link beam
- ASTM A572-50

3. Modelling and Analysis of steel connection in IDEA StatiCa

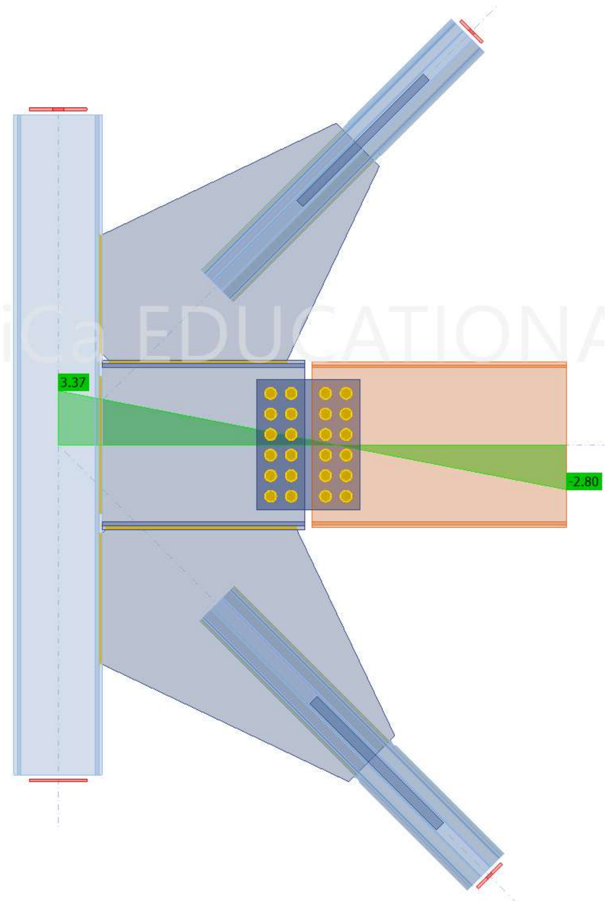
The connection modelling in IDEA StatiCa begins with defining the members- stub beam, link beam and two braces. Then, the two gusset plates and four reinforcing plates on braces are defined and arranged as per connection details. Also, two shear splice plates are defined and assigned at the location as per the detail, away from the connection and connected to the webs of both beams using bolted connection.

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. To perform the CBFEM analysis of the given connection, it was modeled in using several manufacturing operations, for the presented details of connection, and a base model was prepared.

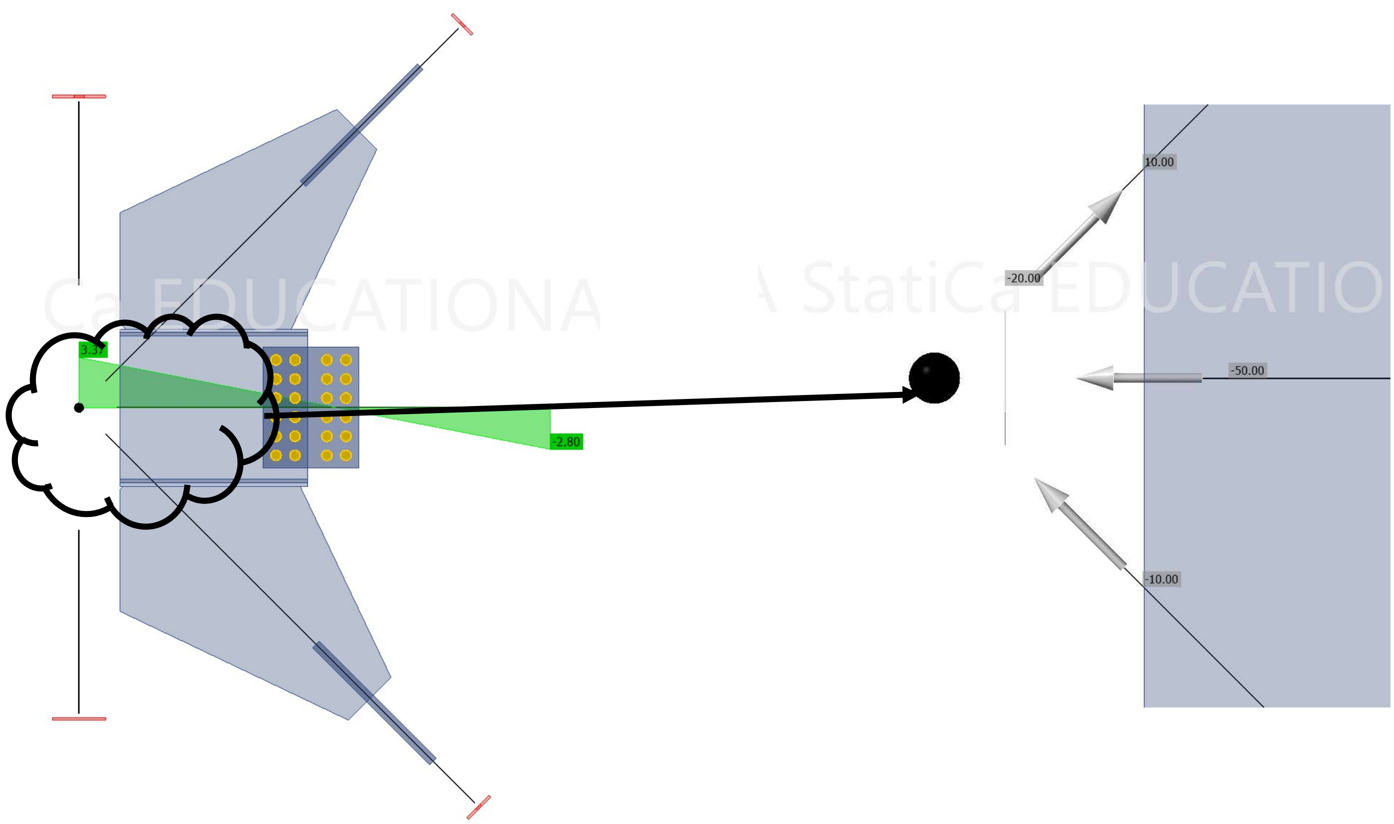
The solid, transparent and wireframe view of the base model is shown in Figure 4. The column is selected as a bearing member in CBFEM. The model type of N-V_y-V_z-M_x-M_y-M_z (Fixed) is assigned to beam and column, while model type of N-V_y-V_z (Pinned) is provided to the two bracing members, such that the forces are in the node, except for beam for which the forces will be in bolts, as seen in wireframe view in Figure 4.



a) Solid view of Base Model in CBFEM



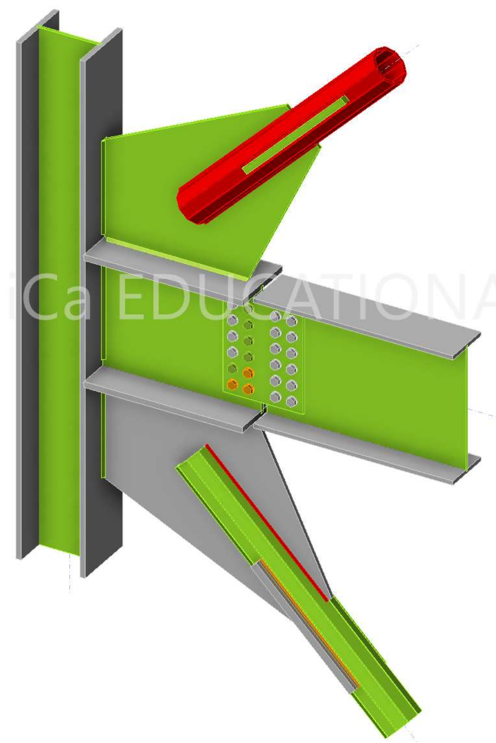
b) Transparent view of Base Model in CBFEM



c) Wireframe view of Base Model in CBFEM

Figure 4 - Configurations of the view of the connection for SCBF – Solid, Transparent and Wireframe view

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 5 and Table 1,



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 5 - Visualization of failure in connection and member in overall check - CBFEM

Based on the observation for the given connection from Figure 5, 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**	Flanges of beam and column, bottom gusset plate, reinforcing plates on bottom brace, several bolts in the shear splice plate.	Elements are safe for action of loads.
2	**GREEN**	Web of column and beam, reinforcing plates on top brace, gusset plate above beam, shear splice plate and bolts in it, welds at column connecting to beam and gusset plates, welds at beam flanges connecting to gusset plate.	Elements are safe for action of loads.
3	**ORANGE**	Welds at bottom brace connecting to reinforcing plates, some bolts in shear splice plate.	Elements are on the verge of failure, for the applied loads.

4	**RED**	Top brace, weld in top brace connecting to reinforcing plates.	Elements are failing for the applied loads; utilization is beyond 100%.

4. Verification of resistance in CBFEM – Capacity of Elements (Plates and Members)

In the previous CBFEM evaluation example for SCBF, we had two braces above the beam and two braces below, such that the brace sizes at each side (above beam and below beam) are the same. Therefore, the direction of loading due to such brace orientation did not affect the connection design to a larger extent. While for the case of this corner connection example, as the braces above and below the beam are of different sizes, the direction of loading considered affects the amount of force that is to be considered in the connection design significantly. This gives rise to two design cases (DC) and two load sets (LS) for each design case for the check for design loads – DC1(Design case 1) and DC2 (Design case 2)

These two design cases are considered as per LRFD for the connection design as per the requirements of AISC Seismic Provisions Section F2.3 and presented in Table 2 and Figure 7.

As per AISC Seismic Provisions Sections F2.3(a) and F2.3(b), two plastic mechanism analyses are required to be considered for the determination of required strengths of beams, columns, and connections. The AISC Seismic Provisions Section F2.6c specifies the required strength of brace connections.

Table 2 – Design load cases for the considered corner connection (AISC 341, 2016)

Design Case (DC)	Required strength of brace connections as per	Load Set (LS)	Axial Tensile Load (kips)	Axial Compressive Load (kips)
DC1	AISC Seismic Provisions Section F2.3(a)	LS1	+560 kips (in brace above beam)	-524 kips (in brace below beam)
	AISC Seismic Provisions Section F2.3(b)	LS 2	+560 kips (in brace above beam)	-105 kips (in brace below beam)
DC2	AISC Seismic Provisions Section F2.3(a)	LS 1	+616 kips (in brace below beam)	-449 kips (in brace above beam)
	AISC Seismic Provisions Section F2.3(b)	LS 2	+616 kips (in brace below beam)	-135 kips (in brace above beam)

As per structural analysis, Design Case 1 was found to be governing due to the following reasons,

- High magnitude of total maximum vertical shear (kips) at beam.
- High magnitude of axial tensile load on brace above the beam.
- High magnitude of axial compressive load on the bottom brace above the beam.

The design loads obtained for the connection for both load cases – Buckling load Case and Post-Buckling load case are as shown in Figure 7. This design loads also account for the beam transfer forces to the opposing beam for the considered load path of seismic forces, which is essential when designing the corner connection in Special Concentrically Braced Frames (SCBFs), wherein the irregular bracing layouts can be encountered as shown in Figure 6.

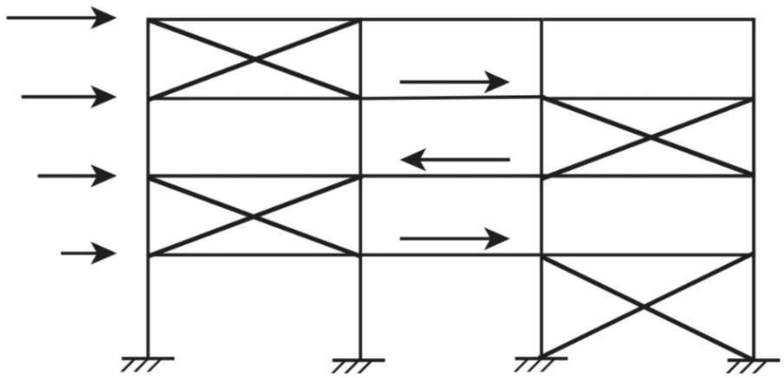
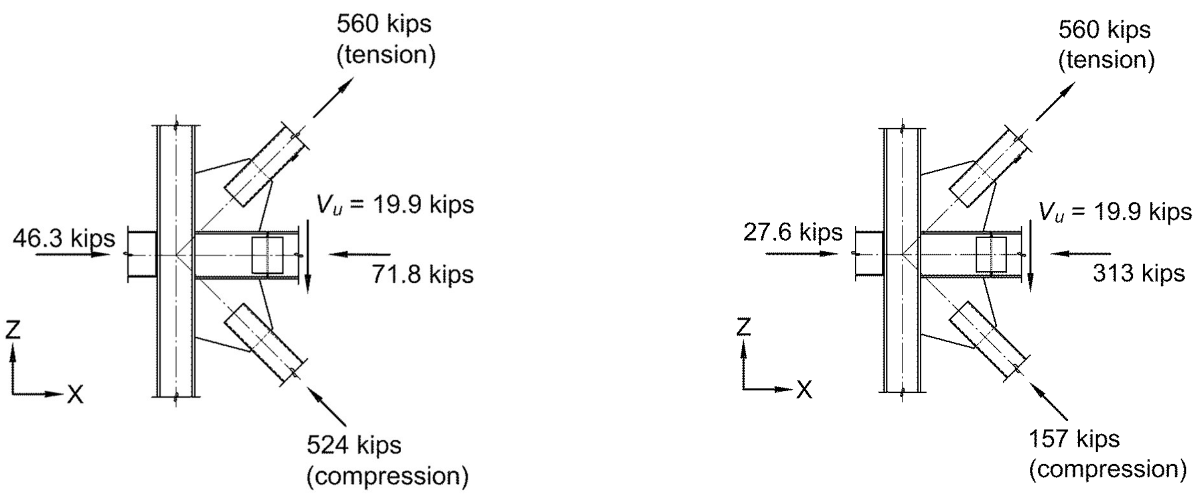
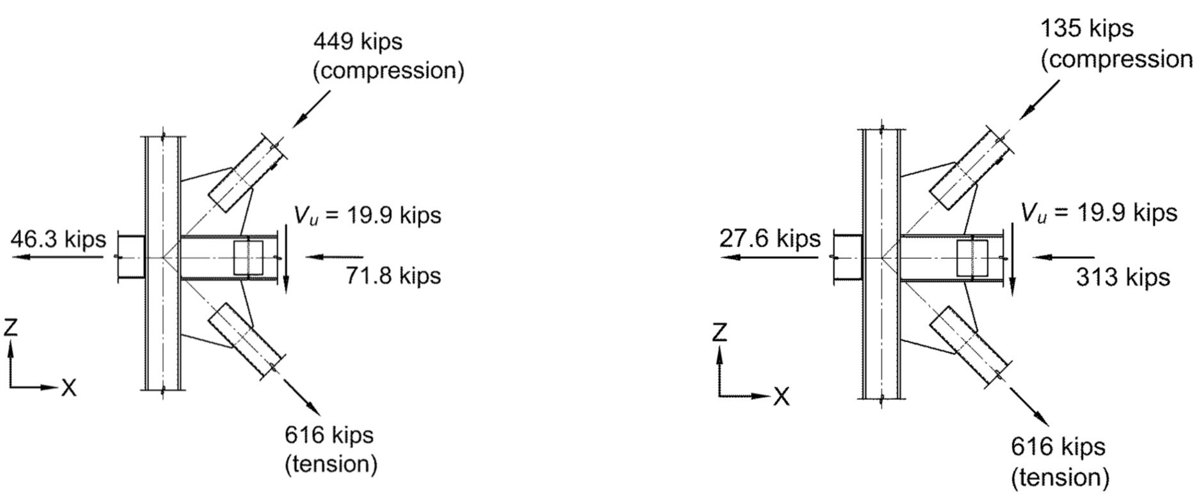


Figure 6 - Forces in transfer beams between incompletely braced bays (Bruneau et al., 2011)



a) Buckling Load Case b) Post-Buckling Load Case

Required strength of brace connections as per AISC Seismic Provisions Section F2.3(a) - LRFD



a) Buckling Load Case b) Post-Buckling Load Case

Required strength of brace connections as per AISC Seismic Provisions Section F2.3(b) - LRFD

Figure 7 - Design load cases for the presented seismic corner connection (AISC 341, 2016)

The “capacity design” analysis was performed for the given connection in CBFEM. Both the brace members are selected as dissipative items. The capacity of elements (plates and members) and connection (bolts and welds) are then evaluated.

The evaluation of capacity for elements – plates and members are performed. The axial loads in all the brace members of the given connection base model were applied starting from 100kips to 1000kips, 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 for both load cases – LC 1 and LC2, as shown in Figures 8 and 9. For load case 1 (LC1), the top brace is subjected to compression load, while the bottom brace is subjected to tension loads. While for the load case 2 (LC2), top brace is subjected to tension load and bottom brace for compression load in CBFEM.

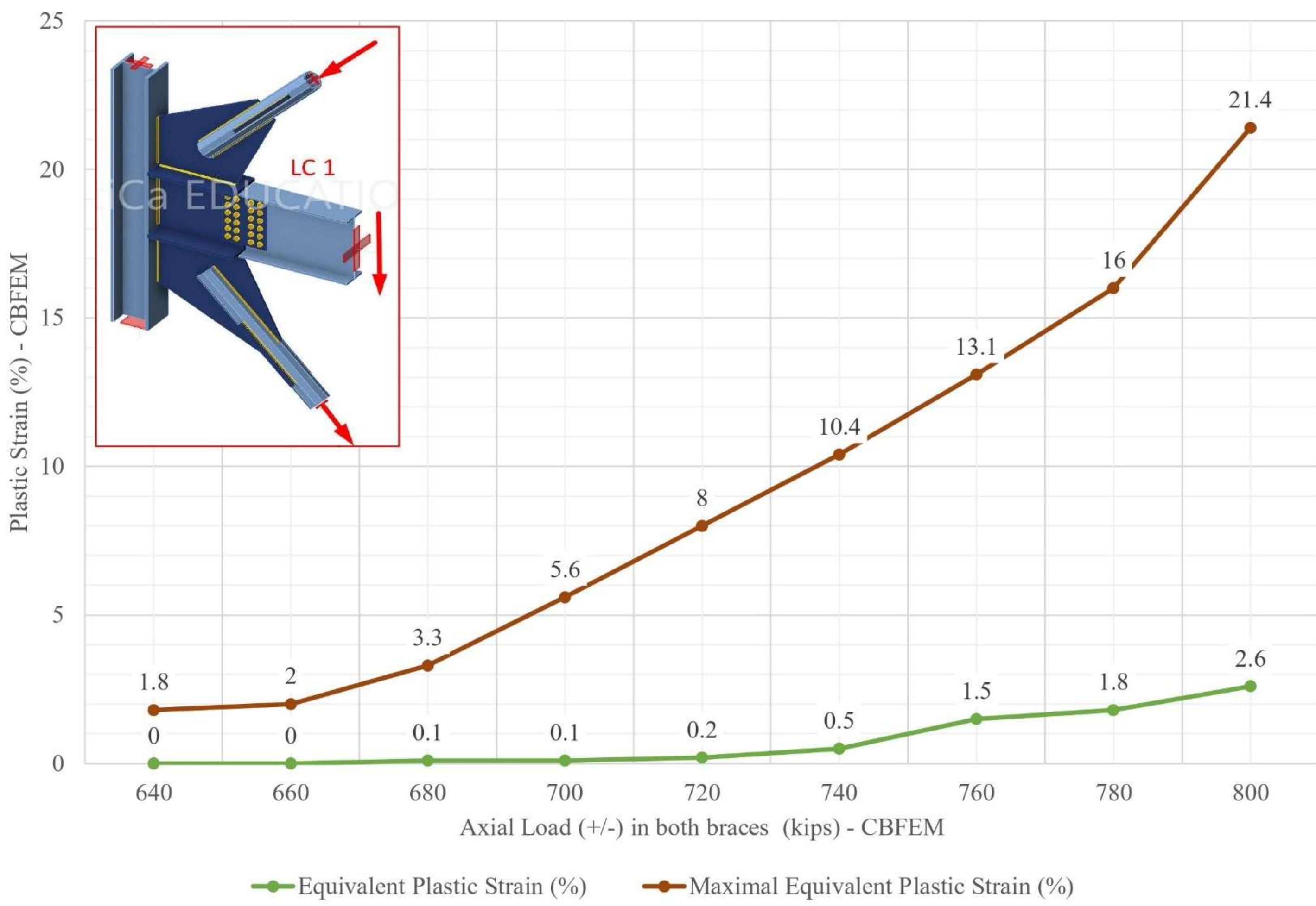


Figure 8 - Graph for evaluation of capacity of elements (plates and members) for LC1 - CBFEM

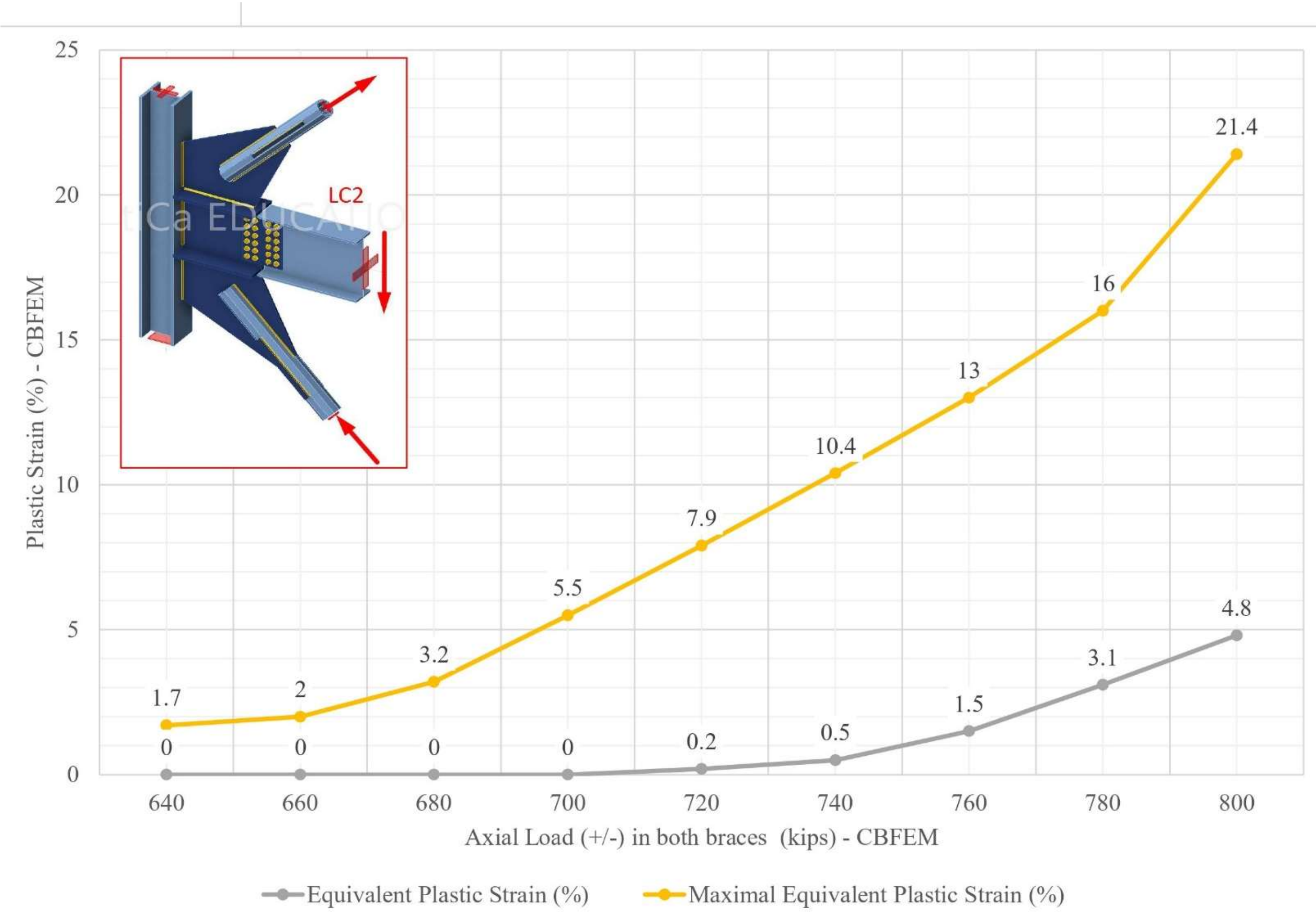


Figure 9 - Graph for evaluation of capacity of elements (plates and members) for LC2 - CBFEM

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. There is a big difference in both these plastic strains for the connection evaluation using "capacity design" analysis and requires further investigation. The same can be confirmed when viewing the results in strain check, as shown in Figure 12.

5. Verification of resistance in CBFEM – Capacity of Connections (Welds and Bolts)

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 and bolted connection at shear splice plate connecting the web of link beam to the beam stub. CJP groove welds, also called demand critical welds, are provided at the beam flanges-to-column location of the presented corner connection.

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 atleast 100% weld utilization at the considered location is observed. Also, the details for the capacity of fillet welds (ϕR_{n_CBFEM}) as per CBFEM and the tension load for which 100% utilization in weld is reached ($P_{Tension\_CBFEM}$) are presented in Table 3 below.

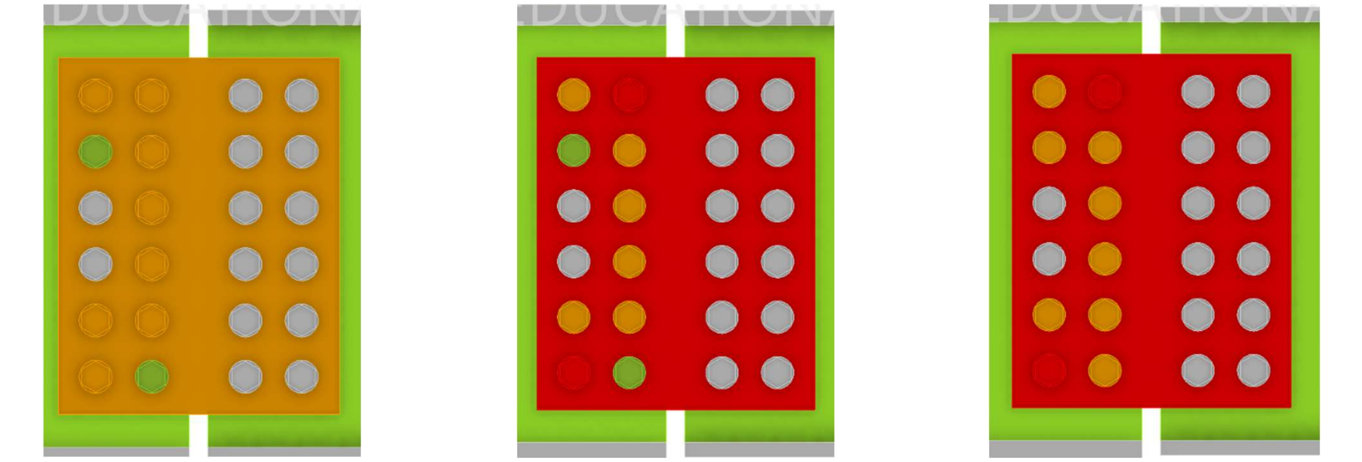
Table 3 – Summary for capacity of fillet welds – CBFEM

Sr No.	Fillet weld connecting	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	Column flange to gusset plate above beam	17	7	1.55	23	483
2	Column flange to gusset plate below beam	19	7	1.57	9	532
3	Reinforcing plate to brace below beam	25	4	0.47	20.3	560
4	Brace member to gusset plate above beam	26	4	0.46	8.3	572
5	Reinforcing plate to brace above beam	26	4	0.47	11.2	516
6	Brace member to gusset plate below beam	28	4	0.46	11.7	629
7	Top flange of beam to gusset plate	26	7	2	25.8	735
8	Bottom flange of beam to gusset plate	28	7	2	76	765

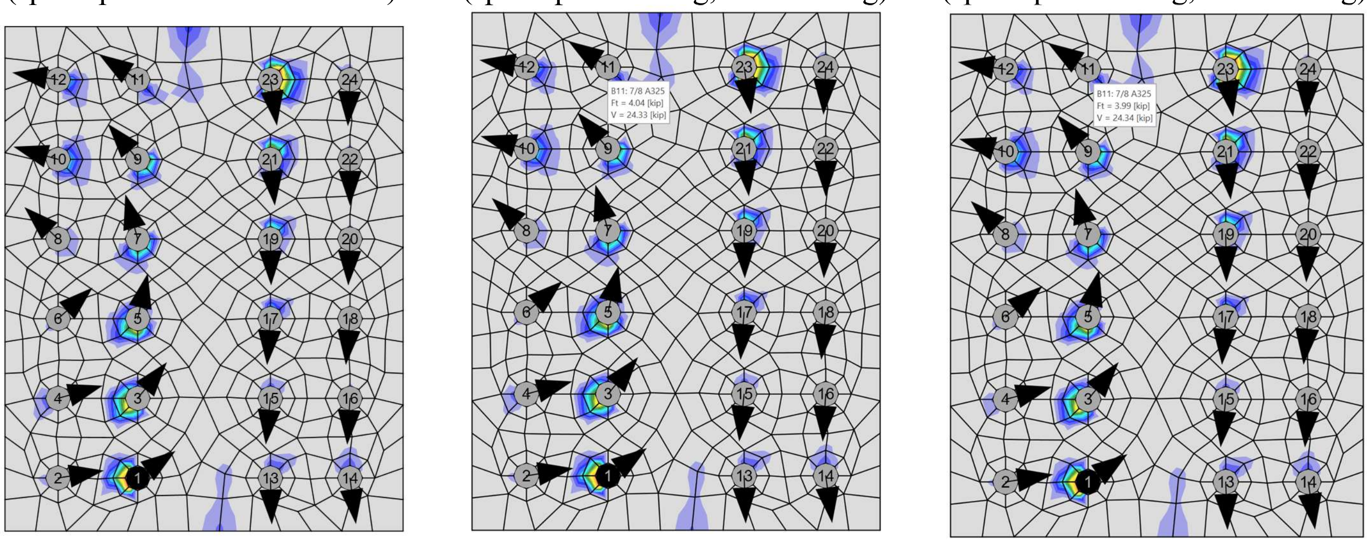
- L_{weld} is the total length of weld in inches as per connection design.
- D_{16} is the weld size, which is specified to the 16th of an inch.
- $L_{weld\_critical}$ is the total length of critical element of weld in CBFEM.
- θ_{CBFEM} is the weld angle in degree as obtained from CBFEM, which is based on the failure of the 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

To evaluate the limit state of shear strength of bolts on splice plate, the vertical shear on the link beam (V_{shear_beam}) is applied starting from 10kips to 150kips, until at least 100% of utilization in bolts is reached. At V_{shear_beam} of 144kips, the limit state of shear strength is observed in bolts as per CBFEM. Also, the splice plate is fail by reaching a plastic strain of more than 5% as shown in Figure 10, with three cases considered. The value for bolt shear strength as per AISC is 124kips which is found to be conservative by 14% when compared to the value from CBFEM that is 144kips.

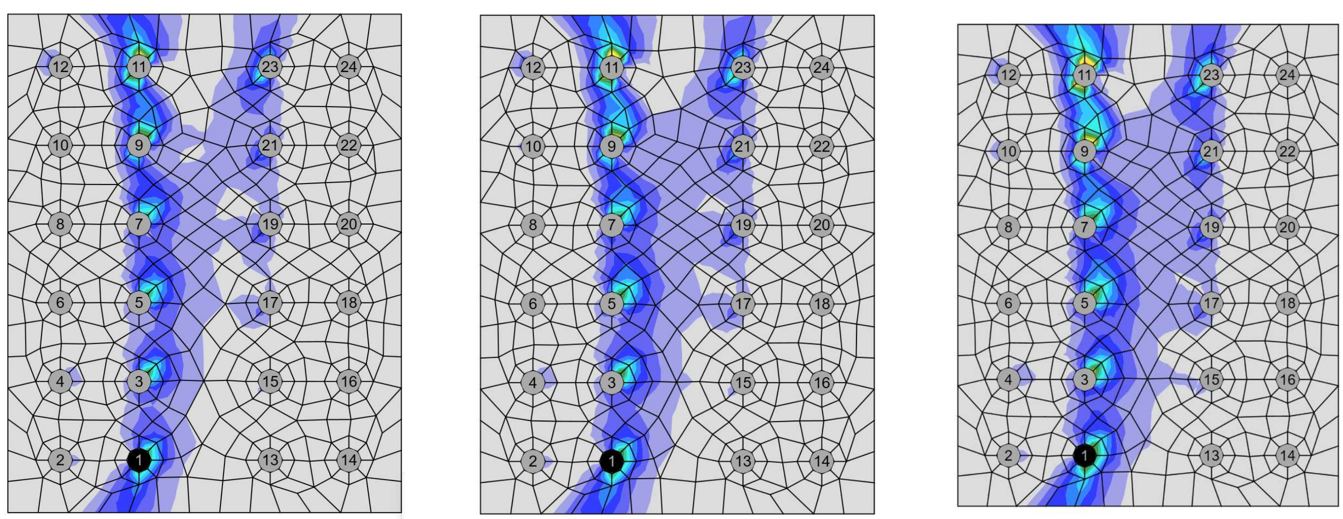
Case 1- V_{shear_beam} = 143kips | Case 2 - V_{shear_beam} = 144kips | Case 3 - V_{shear_beam} = 145kips



a) Overall check – CBFEM (splice plate and bolts are OK) | a) Overall check – CBFEM (splice plate failing, bolts failing) | a) Overall check – CBFEM (splice plate failing, bolts failing)



b) Stresses in contact and bolt forces - CBFEM | b) Stress in contact and bolt forces - CBFEM | b) Stress in contact and bolt forces - CBFEM

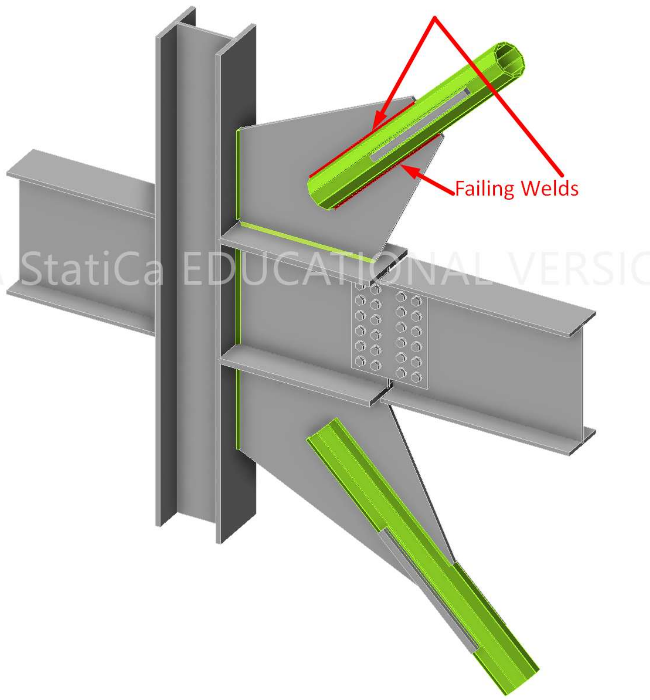


c) Plastic strains in splice plate - CBFEM | c) Plastic strains in splice plate - CBFEM | c) Plastic strains in splice plate - CBFEM

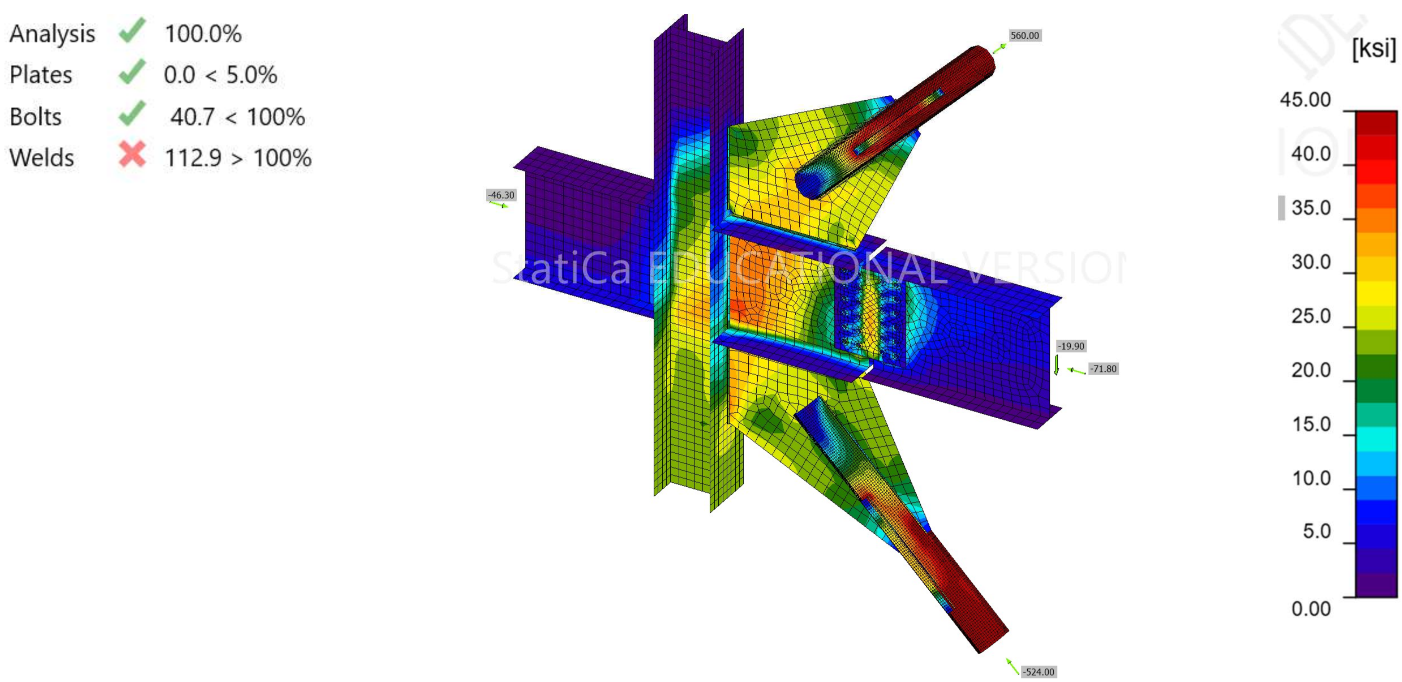
Figure 10 – Evaluation of limit state of shear strength of bolts on splice plate – CBFEM

6. Code check of connection for design loads in CBFEM

The “capacity design” analysis of the given connection is performed for the action of design load cases in CBFEM. Both the brace members are selected as “dissipative item” in capacity design. The biggest advantage of IDEA StatiCa Connection is we can add all the possible load cases for the presented connection, and CBFEM provides us the extreme value amongst the applied loads. The overall check of the given connection for the design load cases – Buckling and post buckling is verified as shown in Figures 11.



a) Overall check of connection for design loads - CBFEM



b) von Mises stresses in connection for design loads - CBFEM

Figure 11 - Evaluation of corner connection for design loads - CBFEM

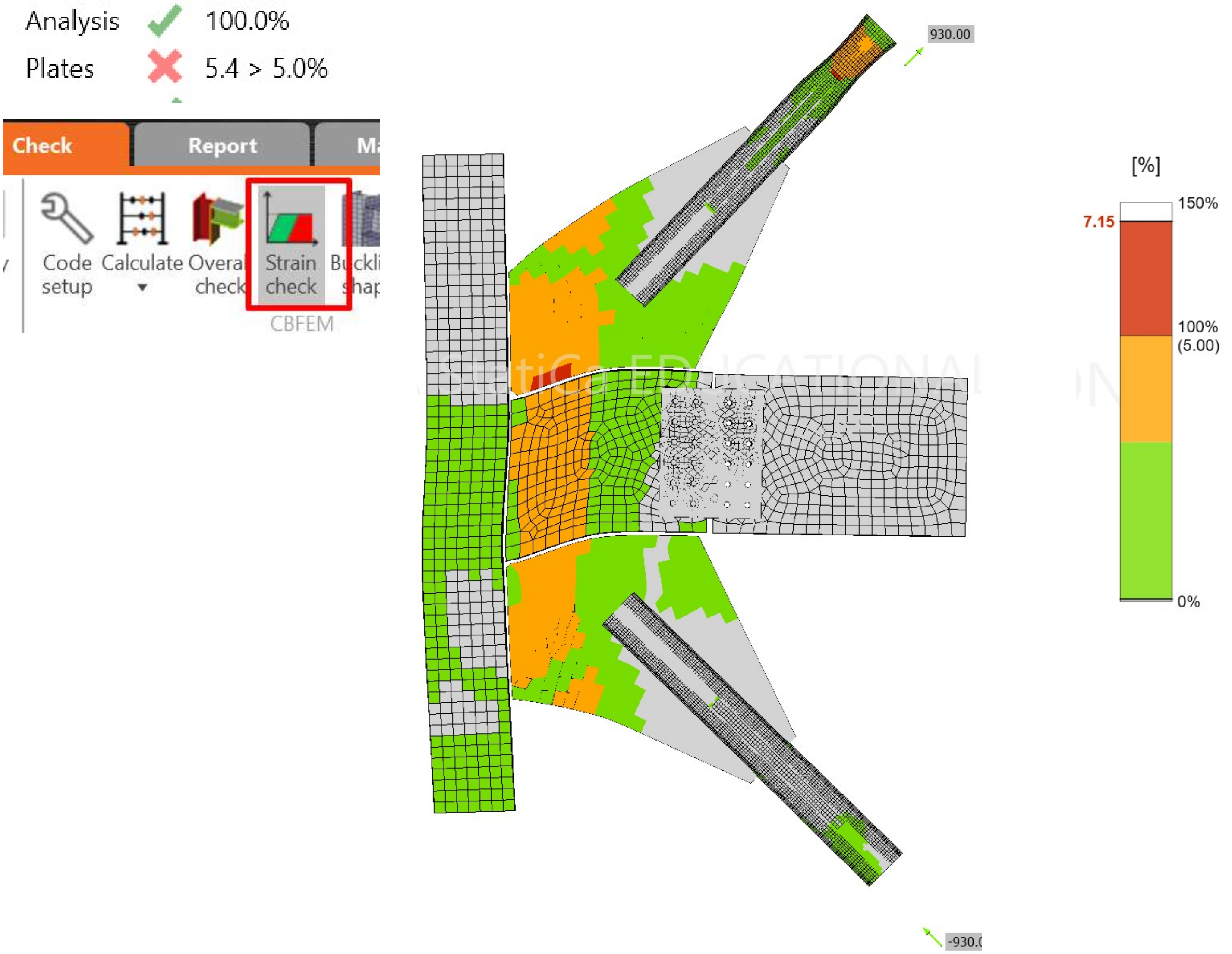
The welded connection between top gusset plate and brace subjected to an axial tension design load of 560kips was observed to be failing with the weld utilization of 113% >100%. The next step a designer must take can be either to increase the weld size to be 5/16” or to increase the weld length, such that the connection is safe, after the seismic analysis of updated CBFEM model.

7. Evaluation of limit states for tension loads in brace - CBFEM

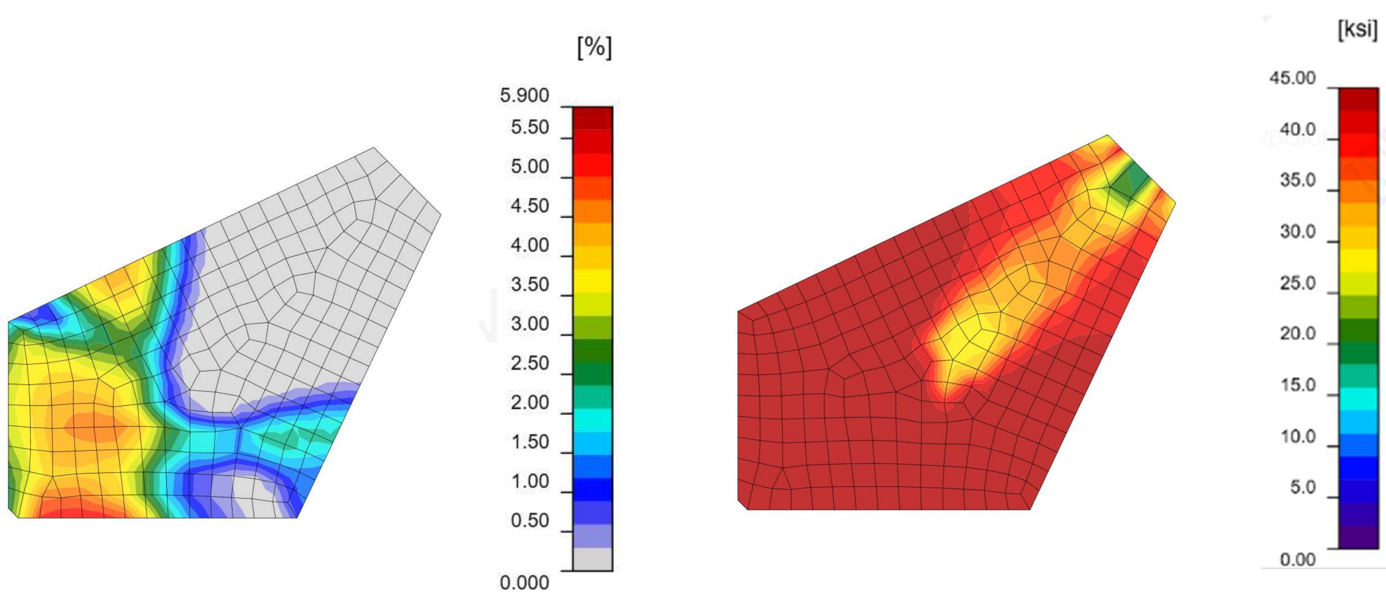
The calculations according to AISC *Specification* (AISC 360, 2016) are performed in accordance with the provisions for load and resistance factor design (LRFD) to obtain the results for several limit states for the presented connection. In CBFEM, the limit states will be investigated individually with several iterations, by observing the plastic strains and von Mises stresses (equivalent stresses), and then the capacities will be reported accordingly from IDEA StatiCa Version 23.1.

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 the study was to evaluate the limit states related to connection only.

The limit state of tensile yielding for gusset plates was observed for an axial load of 930kips in bracing members, as shown in Figure 12. Whereas for the bottom gusset plate, this limit state was observed for an axial load of 990kips in bracing members. When compared to the AISC results, CBFEM yields approximately similar values, but on a higher side with an average difference of 5%.



a) Strain check in connection – CBFEM (Deformed shape factor =5)



b) Plastic strains in top gusset plate - CBFEM c) von Mises stresses in top gusset plate - CBFEM

Figure 12 - Evaluation for limit state of tensile yielding of brace - strain check and plastic strains - 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 code. 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 weld connections.

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 connecting elements. 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 as shown in Figure 13.

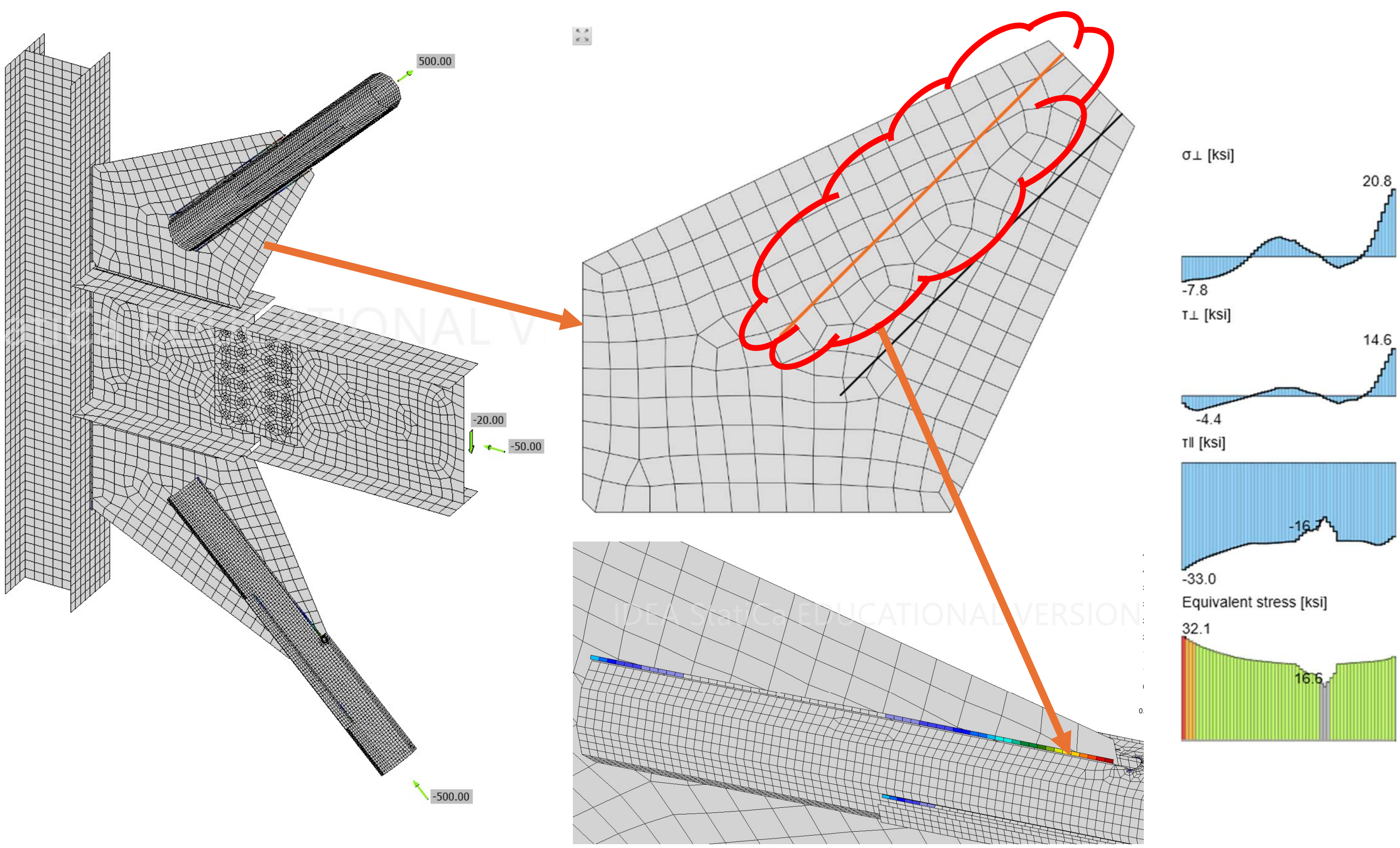


Figure 13: Plastic strains for evaluation for limit states related to welded connection – CBFEM

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. 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 14 and 15.

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

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16ths weld size (in)	
			AISC	CBFEM
1	449	Brace-to-gusset plate above beam	4	5
2	466	Brace-to-gusset plate below beam	4	5
3	663.8	Reinforcing plate-to-brace above beam	4	5
4	720	Column flange-to-gusset plate below beam	7	8
5	723.5	Reinforcing plate-to-brace below beam	4	5
6	842	Bottom flange of beam to gusset plate	7	8
7	895	Top flange of beam to gusset plate	8	9
8	906	Column flange-to-gusset plate above beam	8	9

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

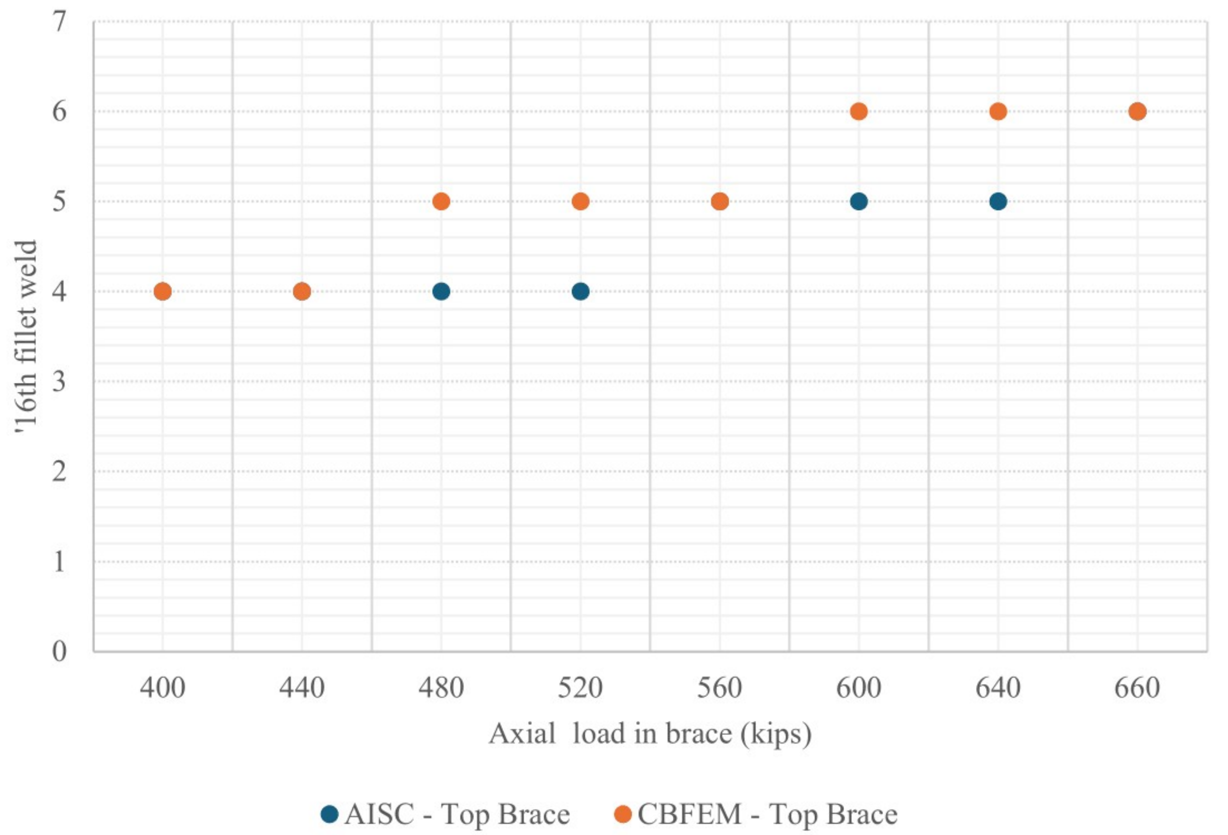


Figure 14: Comparison of required size of fillet weld at brace-to-gusset plate above beam

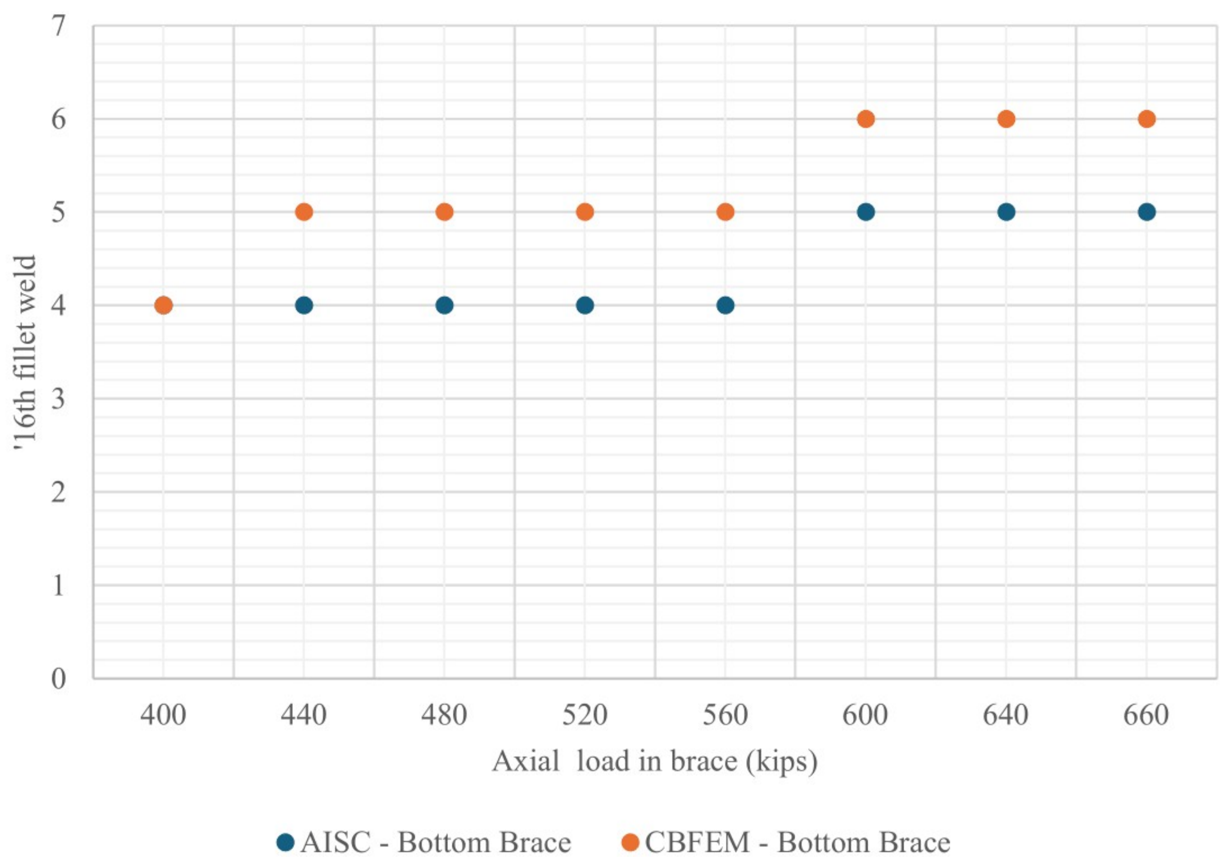


Figure 15: Comparison of required size of fillet weld at brace-to-gusset plate below beam

8. Evaluation of limit state for compression loads in brace - CBFEM

For the brace member subjected to the action of compression loads, the most common strategy to provide rotational flexibility at the brace connection is by accommodating a linear fold line or hinge line of thickness “2t” for out-of-plane buckling, where t is the gusset plate thickness. With this approach, rotation of the brace end is permitted by allowing a line of rotation in the gusset plate for the action of high compression loads in the brace as shown in Figure 16.

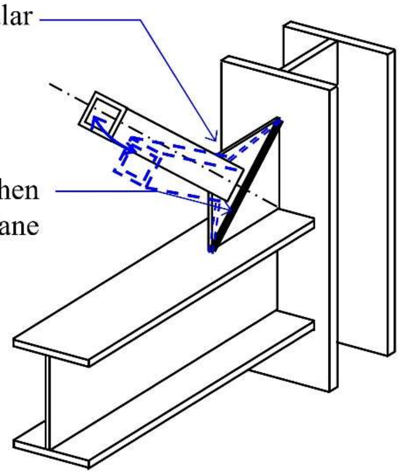
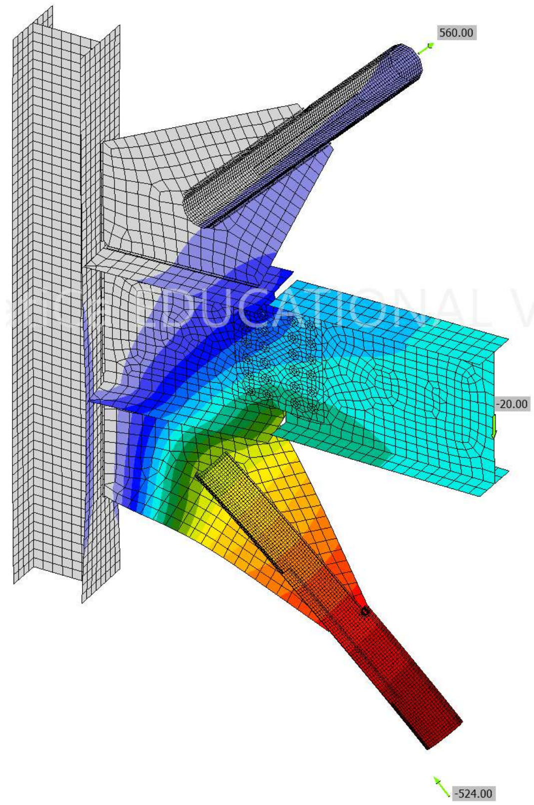


Figure 16: Accommodation of out-of-plane brace buckling in SCBF (AISC 341, 2016)

The evaluation of buckling limit state for this connection is performed for two conditions, For the first condition, buckling evaluation of bottom brace is performed. Design Case 1 and Load Set 1 are considered, such that the axial tension load in the brace above beam is 560kips and axial compression load in the brace below the beam is -524kips as shown in Table 2. While for the second condition, buckling evaluation of top brace is performed. Design Case 2 and Load Set 1 are considered, such that the axial compression load in the brace above beam is 449kips and axial tension load in the brace below the beam is 616kips as shown in Table 2.

Using the buckling multiplier factor and mode shapes obtained in CBFEM, we can check the buckling limit state requirement as per AISC for the gusset plate in IDEA StatiCa Connection. As per the results from CBFEM shown in Figure 17, the buckling factor for critical mode (mode1) is 2.14 for an axial compressive load of 560 kips in brace and observed in the brace member. The entire gusset plate is restrained on two sides. One is the weld connection to the beam and the is the weld connection to the column flange. Hence, it is concluded that the category of the buckling issue for this connection is local buckling. For local buckling category of gusset plate, since the obtained buckling factor of 2.14 for critical mode is conservative than the required buckling factor of 4, and the buckling mode shape is observed in the brace member, the given connection requires further investigation for non-linear buckling analysis using IDEA StatiCa member. Similar observations are found for the evaluation of connection for buckling in condition 2

Analysis	✓	100.0%
Plates	✓	0.0 < 5.0%
Bolts	✓	17.1 < 100%
Welds	✗	113.4 > 100%
Buckling		2.14

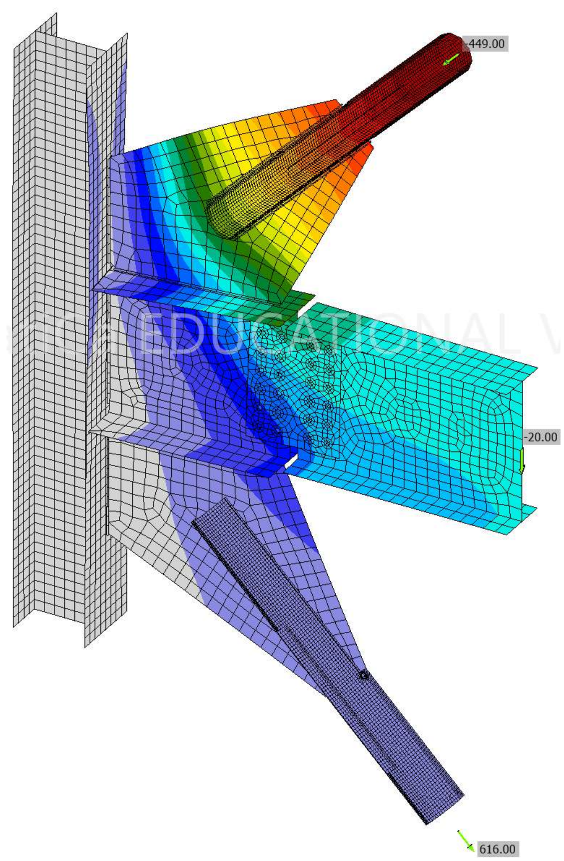


Local buckling of joint

	Loads	Shape	Factor
>	DC1 LS1	1	2.14
		2	3.65
		3	6.74

a) Condition 1 – Design Case 1 and Load Set 1 from Table 3 (Deformed shape factor = 20)

Analysis	✓	100.0%
Plates	✓	0.0 < 5.0%
Bolts	✓	34.8 < 100%
Welds	✗	112.1 > 100%
Buckling		2.38



Local buckling of joint

	Loads	Shape	Factor
>	DC2 LS1	1	2.38
		2	4.09
		3	8.64

b) Condition 2 – Design Case 2 and Load Set 1 from Table 3 (Deformed shape factor = 20)

Figure 17: Evaluation of buckling limit state for corner connection - CBFEM

9. Parametric Studies

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

The failure of fillet welds at brace-to-gusset was observed as the governing limit states for the action of design loads. Therefore, the fillet welds at all reinforcing plate-to-brace and brace-to-gusset plate locations are changed in the design model to be full-penetration butt welds (CJP welds). Based on this modification, the load at braces can now be increased up to 835 kips before a limit state of weld is observed at beam stub-to-bottom gusset plate from results as shown in Figure 18. This is a 50% increase in the load carrying capacity for the presented corner connection as per CBFEM.

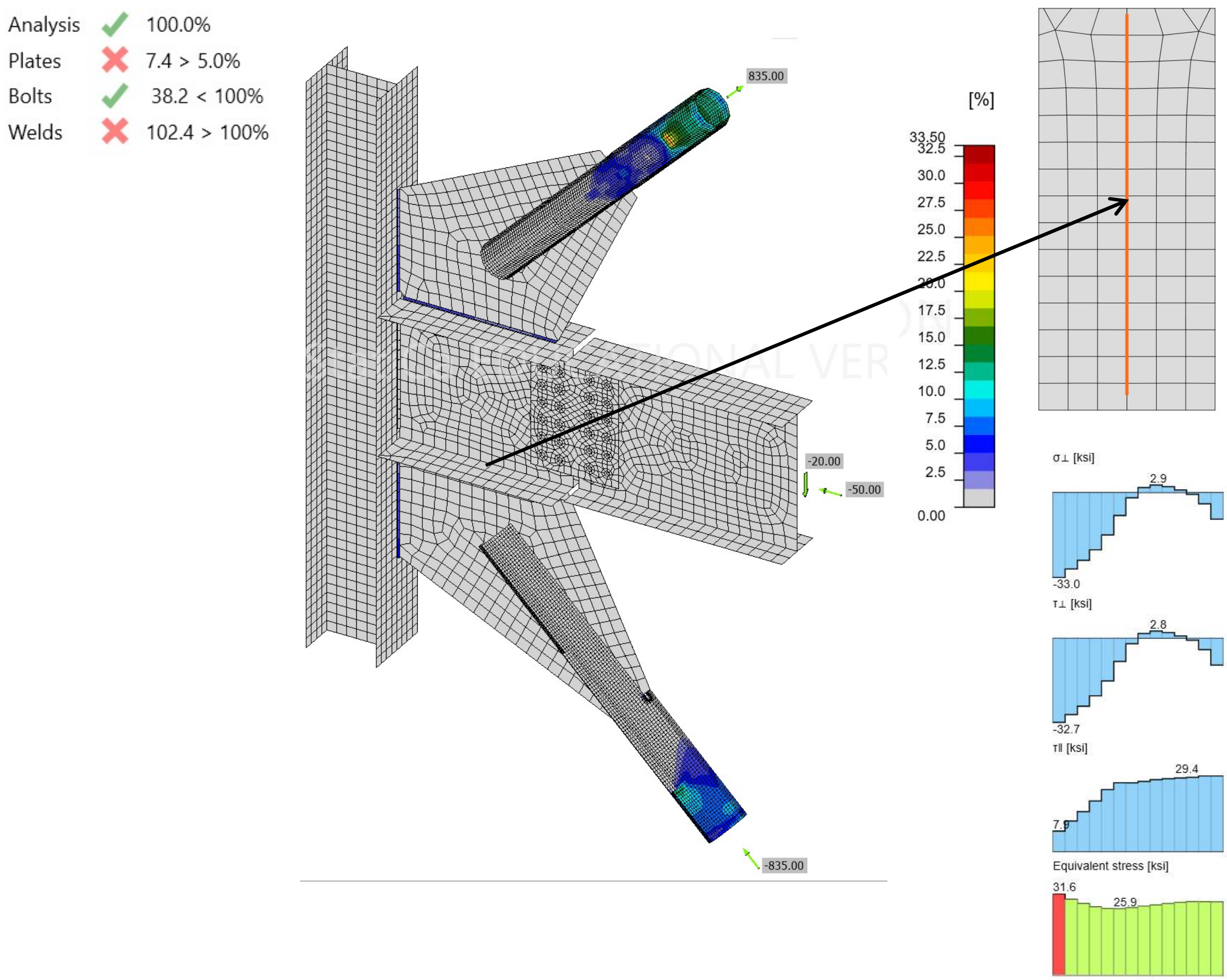
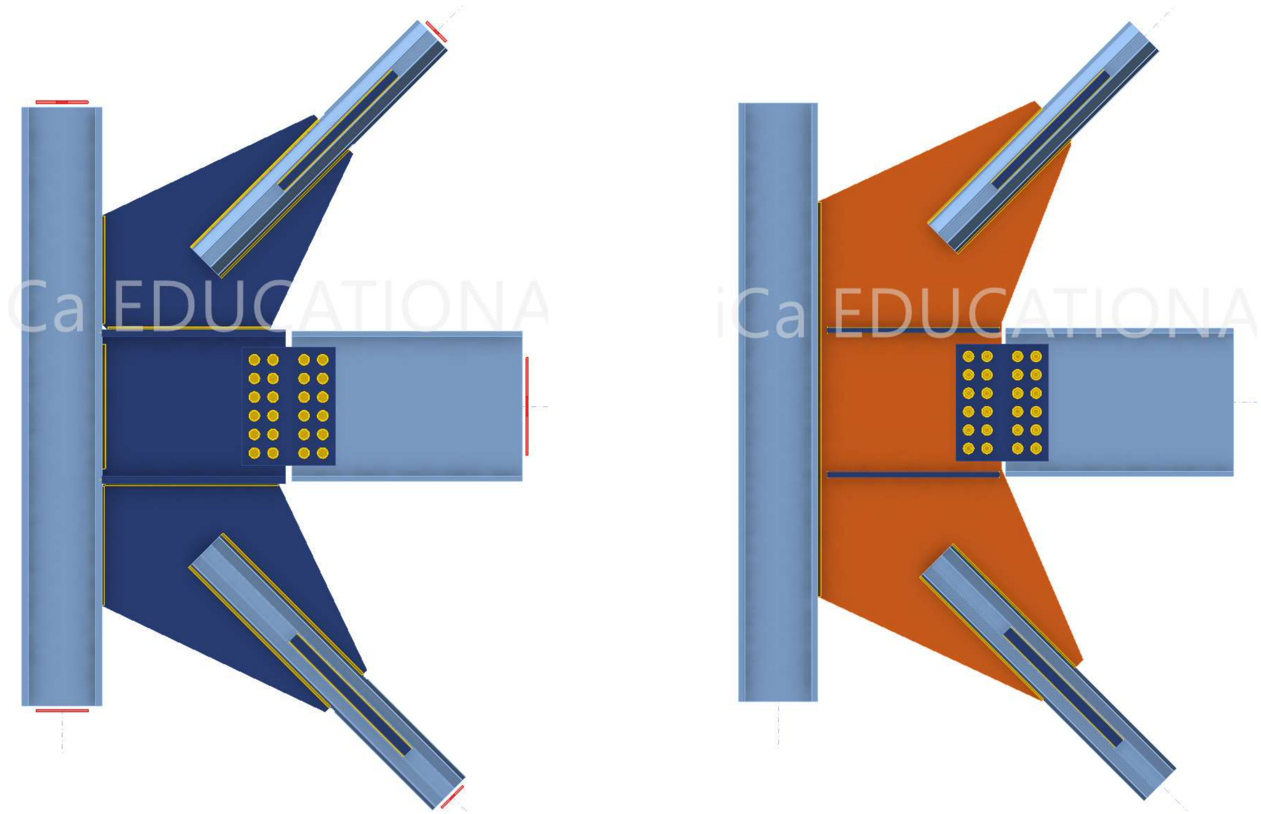


Figure 18: Plastic strain for change of fillet welds at all reinforcing plate-to-brace and brace-to-gusset plate locations - CBFEM

b. Comparison to alternative connection design

An alternate connection detail using a continuous gusset plate, replacing the beam stub was considered in CBFEM for this parametric study as shown in Figure 19. This detail utilizes a 3/4in thick continuous gusset plate with plate reinforcement in lieu of the W24x146 beam stub and thereby eliminates many welds, hence providing an economical alternate solution.



a) Example 5.3.9, AISC 341-16

b) Alternate detail using a continuous gusset plate, AISC 341-16

Figure 19: Comparison to alternate corner connection design utilizing a continuous gusset plate - CBFEM

As the limit state of weld connection was observed to be governing one, the same was evaluated in detail. Overall, the alternate detail was found to have slightly less weld utilization in CBFEM which means more weld capacity, with an average difference of around 5% in results with the connection having single gusset plate in each side as shown in Figure 20.

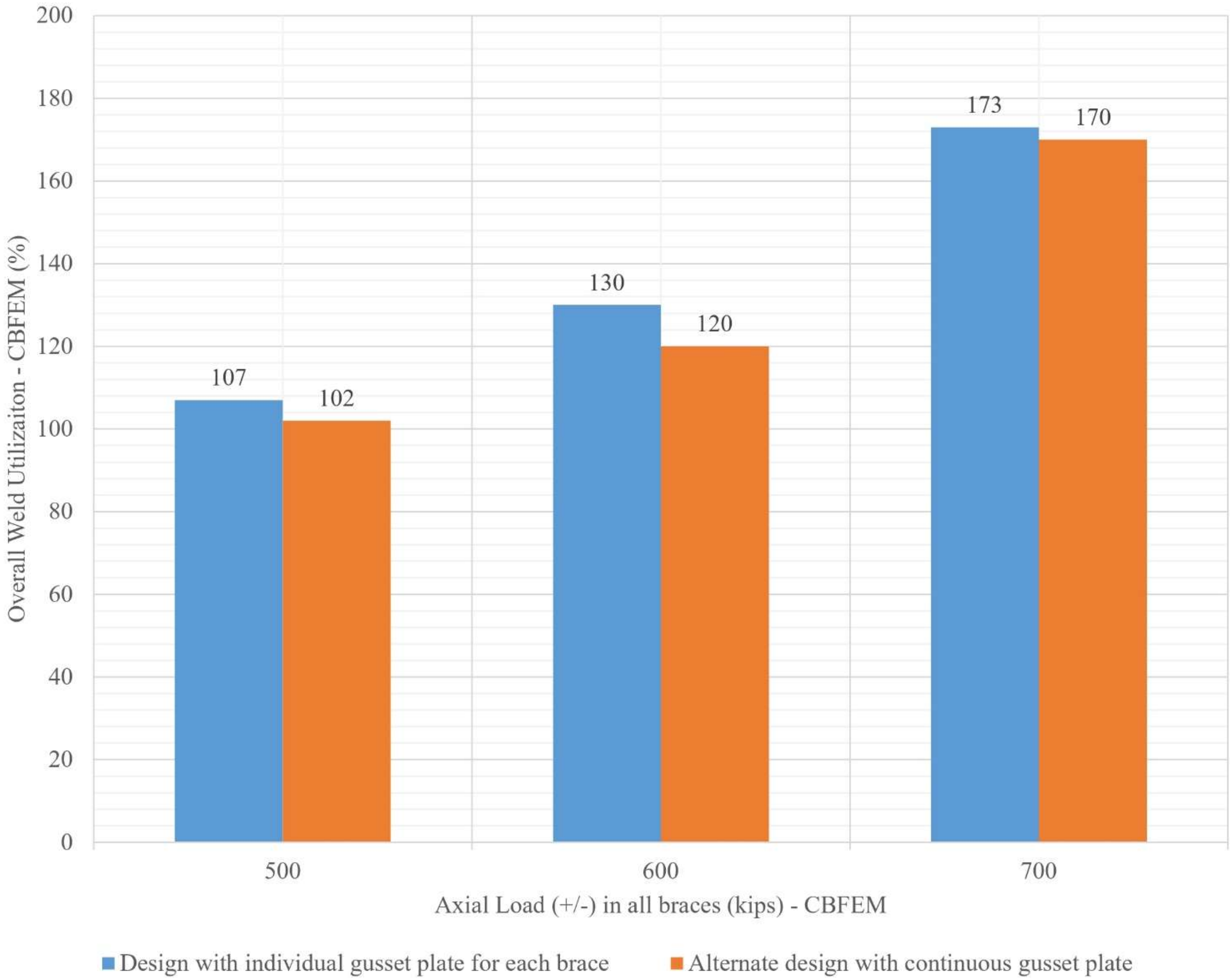


Figure 20: Comparison of overall weld utilization for alternate design

When evaluating the shear splice plate and bolted connection, vertical shear in the end of link beam is increased starting from 10kips, until one of the following is observed – at least 5% plastic strains in shear splice plate or / and at least 100% utilization of bolts connecting the shear splice plates to beam and gusset plate. Figure 21 represents comparative results, wherein the alternate design is observed to have an excellent edge over the design involving the single gusset plate for each brace. This can be attributed to the single continuous thick gusset plate which acts as an excellent restraint for the vertical shear loads, thereby transferring all the shear to the column web effectively as seen in Figure 22 and providing better energy dissipation overall.

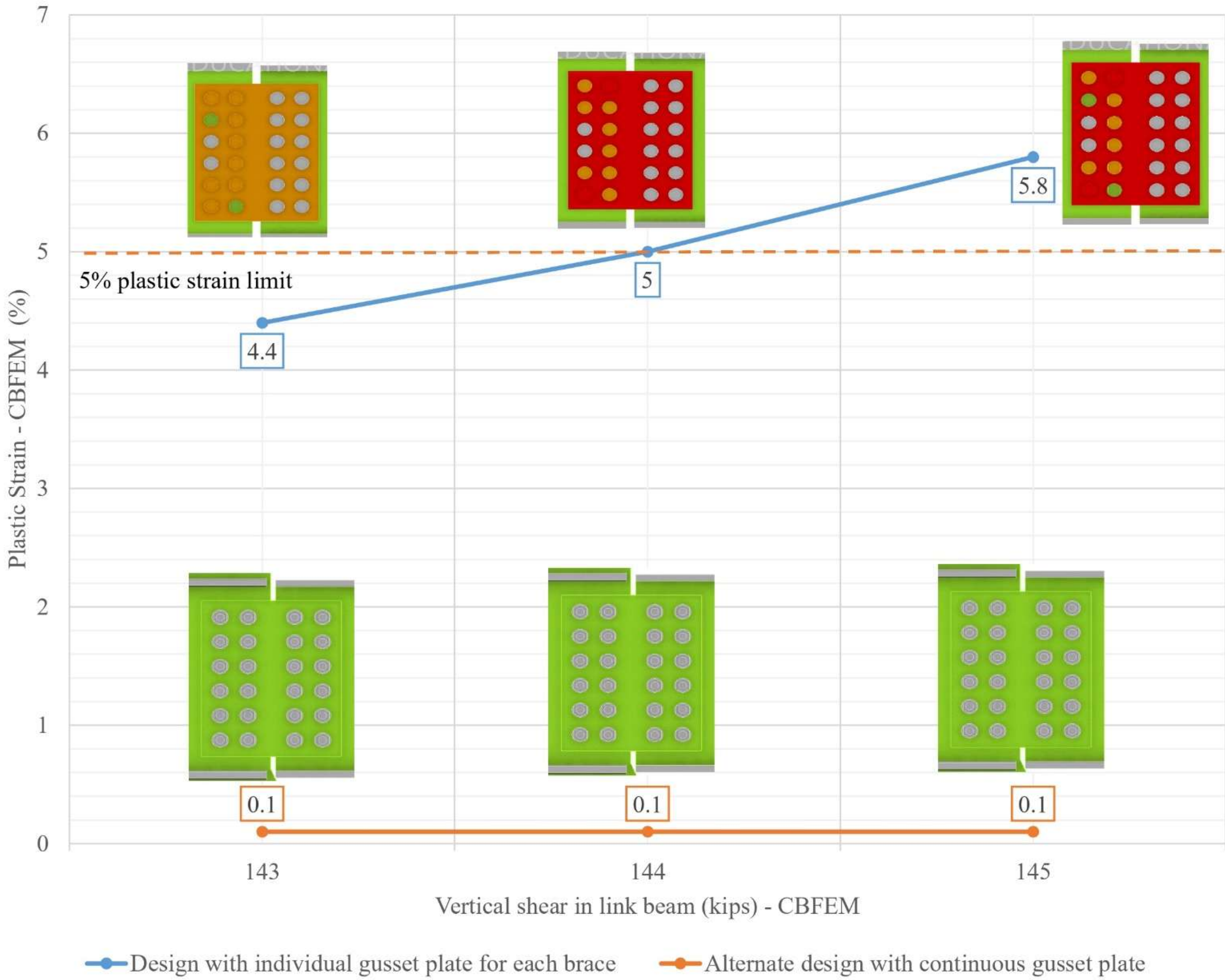
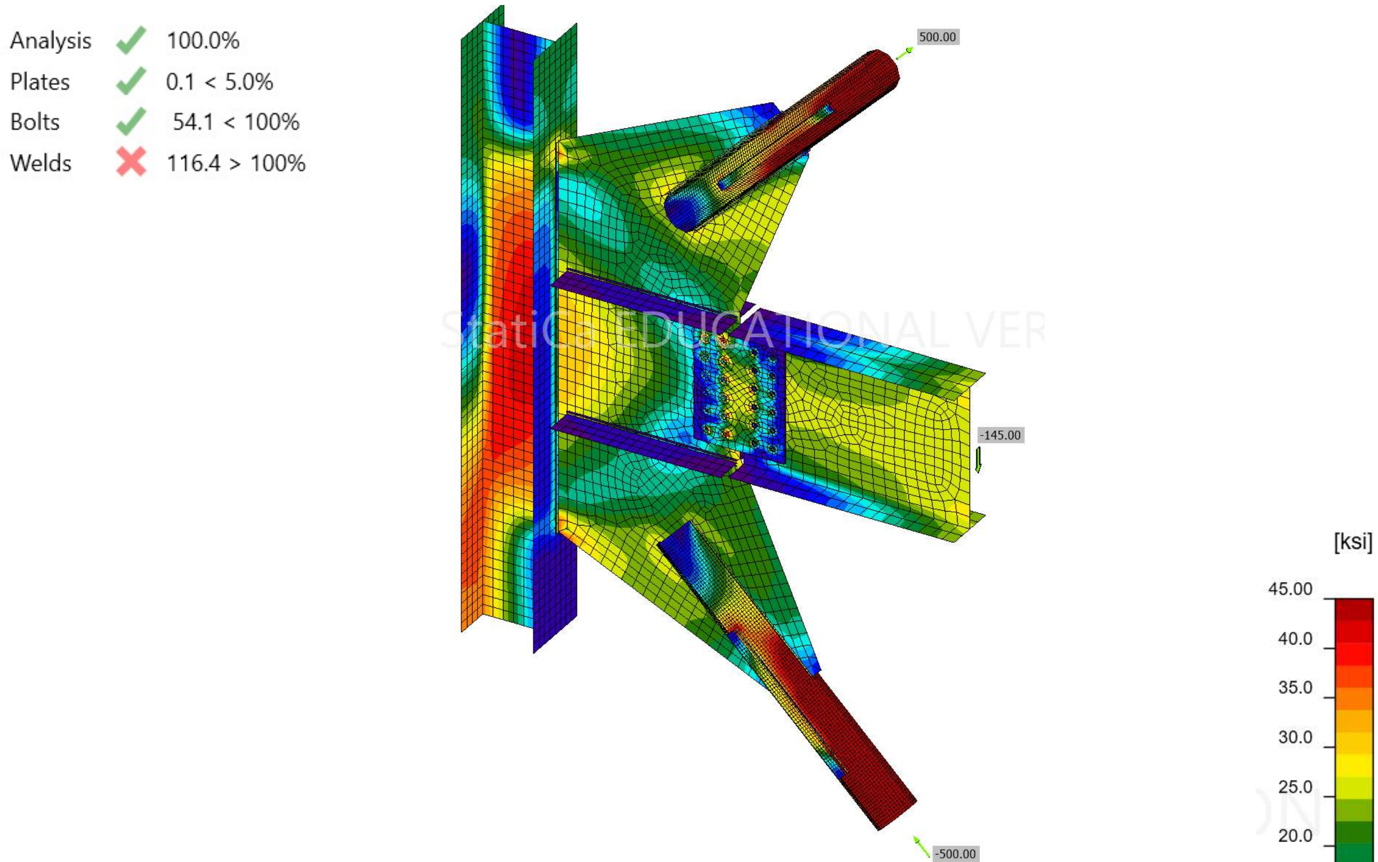
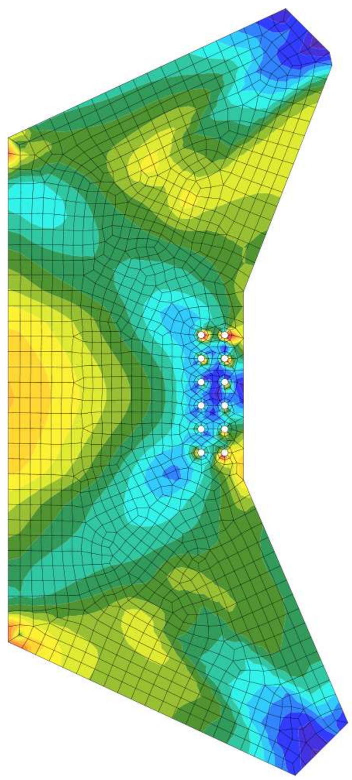


Figure 21 – Plot of vertical shear in link beam vs plastic strain in CBFEM



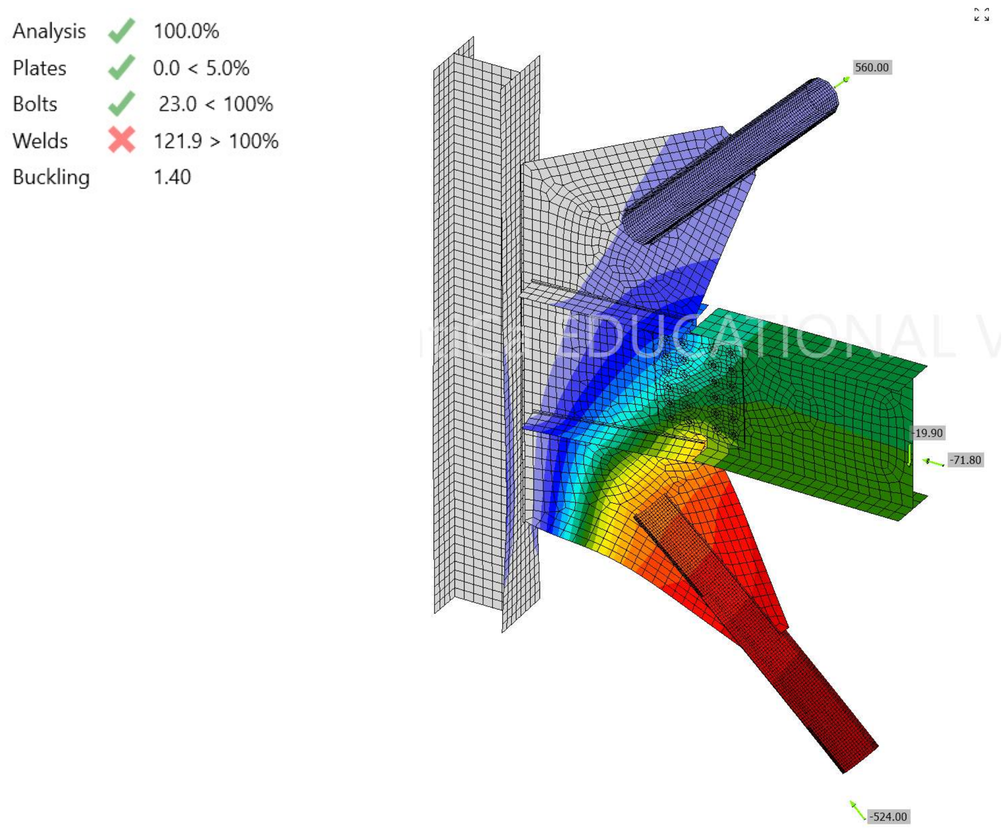
a) von Mises stresses in connection for evaluation of shear splice plate and bolted connection for alternate design - CBFEM



b) von Mises stresses in continuous gusset plate and bolt holes for bolted connection - CBFEM

Figure 22: Evaluation of alternate design for corner connection - CBFEM

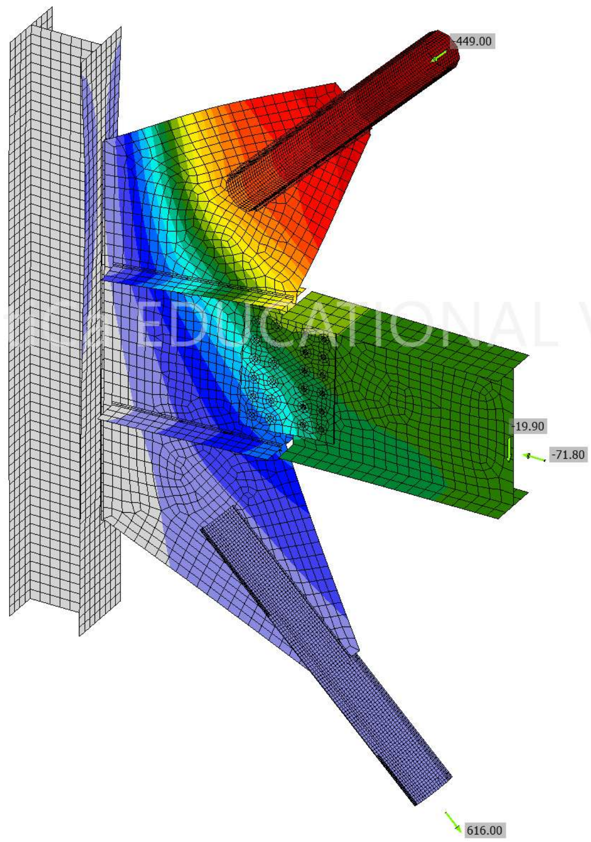
For the case of buckling analysis, the governing load case for each brace as given Table 2 is considered. As per the results from CBFEM shown in Figure 23, the buckling factor for critical mode (mode1) is 1.4 for an axial compressive load of 524kips in brace and observed in the brace member. The entire gusset plate is restrained on two sides. One is the weld connection to the beam and the is the weld connection to the column flange. Hence, it is concluded that the category of the buckling issue for this connection is "local buckling". For local buckling category of gusset plate, since the obtained buckling factor of 1.4 for critical mode is conservative than the required buckling factor of 4, and the buckling mode shape is observed in the brace member. Therefore, the given connection requires further investigation for non-linear buckling analysis using IDEA StatiCa member. Similar observations are found for the evaluation of connection for buckling in condition 2



	Loads	Shape	Factor
>	DC1 LS1	1	1.40
		2	3.16
		3	4.53

a) Condition 1 – Design Case 1 and Load Set 1 from Table 3 to evaluate buckling for bottom brace – CBFEM (Deformed shape factor = 20)

Analysis	✔ 100.0%
Plates	✔ 0.1 < 5.0%
Bolts	✔ 17.5 < 100%
Welds	✘ 125.0 > 100%
Buckling	1.53



Local buckling of joint

	Loads	Shape	Factor
>	DC2 LS1	1	1.53
		2	3.52
		3	5.14

b) Condition 2 – Design Case 2 and Load Set 1 from Table 3 to evaluate buckling for top brace – CBFEM (Deformed shape factor = 20)

Figure 23: Evaluation of alternate design for buckling - CBFEM

It can be concluded that the alternate design consisting of continuous gusset plate is the effective design for the case of this corner connection. This continuous gusset plate can be challenging to be fabricated in shop and erected in field, owing to the larger vertical dimension of gusset. Therefore, it’s essential to co-ordinate with erectors and fabricators for the connection type preferences, before finalizing a detail.

10. Flowcharts

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

11. Comparison of limit state values from CBFEM with AISC

Sr No	Limit State of	AISC (kips)	CBFEM (kips)	Δ_{AISC_CBFEM} (%)
1	Tensile yielding of top gusset plate	871	930	+6.3
2	Tensile yielding of bottom gusset plate	945	990	+4.5

Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16^{ths} weld size (in)	
			AISC	CBFEM
1	449	Brace-to-gusset plate above beam	4	5
2	466	Brace-to-gusset plate below beam	4	5
3	663.8	Reinforcing plate-to-brace above beam	4	5
4	720	Column flange-to-gusset plate below beam	7	8
5	723.5	Reinforcing plate-to-brace below beam	4	5
6	842	Bottom flange of beam to gusset plate	7	8
7	895	Top flange of beam to gusset plate	8	9
8	906	Column flange-to-gusset plate above beam	8	9

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

12. 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 6 and 7 for the case of capacity design, which was approximately 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.

13. Suggestions for Improvement

1. Joint Design Resistance for seismic vertical bracing connections.
2. “Loads in equilibrium” not an option in capacity design as of V23.1.

14. References

- AISC 341. (2016). *Seismic Design Manual. American Institute of Steel Construction, Chicago, Illinois.*
- AISC 360. (2016). *Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago, Illinois.*
- Bruneau, M., Uang, C.-M., & Sabelli, R. (2011). *Ductile Design of Steel Structures* (2nd Edition). McGraw-Hill Education. https://www.accessengineeringlibrary.com/content/book/9780071623957