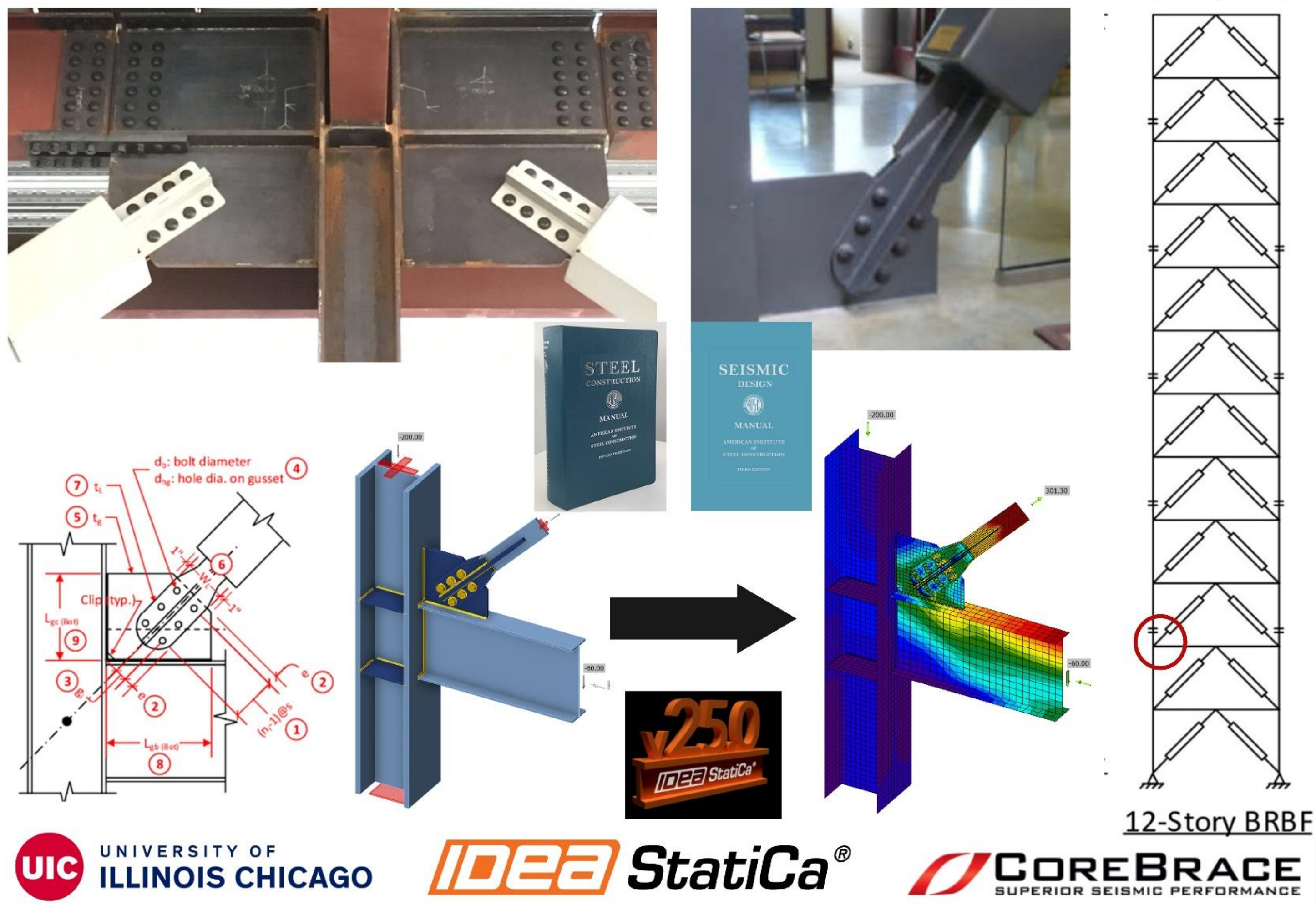
Evaluation of Bolted-Type CoreBrace Corner Connection in Twelve-Story Building of Buckling Restrained Braced Frame (BRBF) System (AISC 360 and 341)



Cover Image

This report presents the second verification study in the seismic analysis and design of bracing connections in Buckling-Restrained Braced Frames (BRBFs), focusing on the evaluation of a bolted-type CoreBrace Corner BRB connection. The investigation compares results obtained using the traditional design approach based on AISC360-16, following AISC 341-16 provisions, with those derived from the Component-Based Finite Element Method (CBFEM) through inelastic analysis performed in IDEA StatiCa Connection V25.0, released in April 2025.

This work was conducted by Dr. Mustafa Mahamid and Satyam Bhosale at the University of Illinois at Chicago. The authors gratefully acknowledge the engineering team at CoreBrace for providing detailed design and analysis data for BRB connections. These inputs were critical in developing Revit 2025 connection sketches and generating accurate CBFEM models in IDEA StatiCa.

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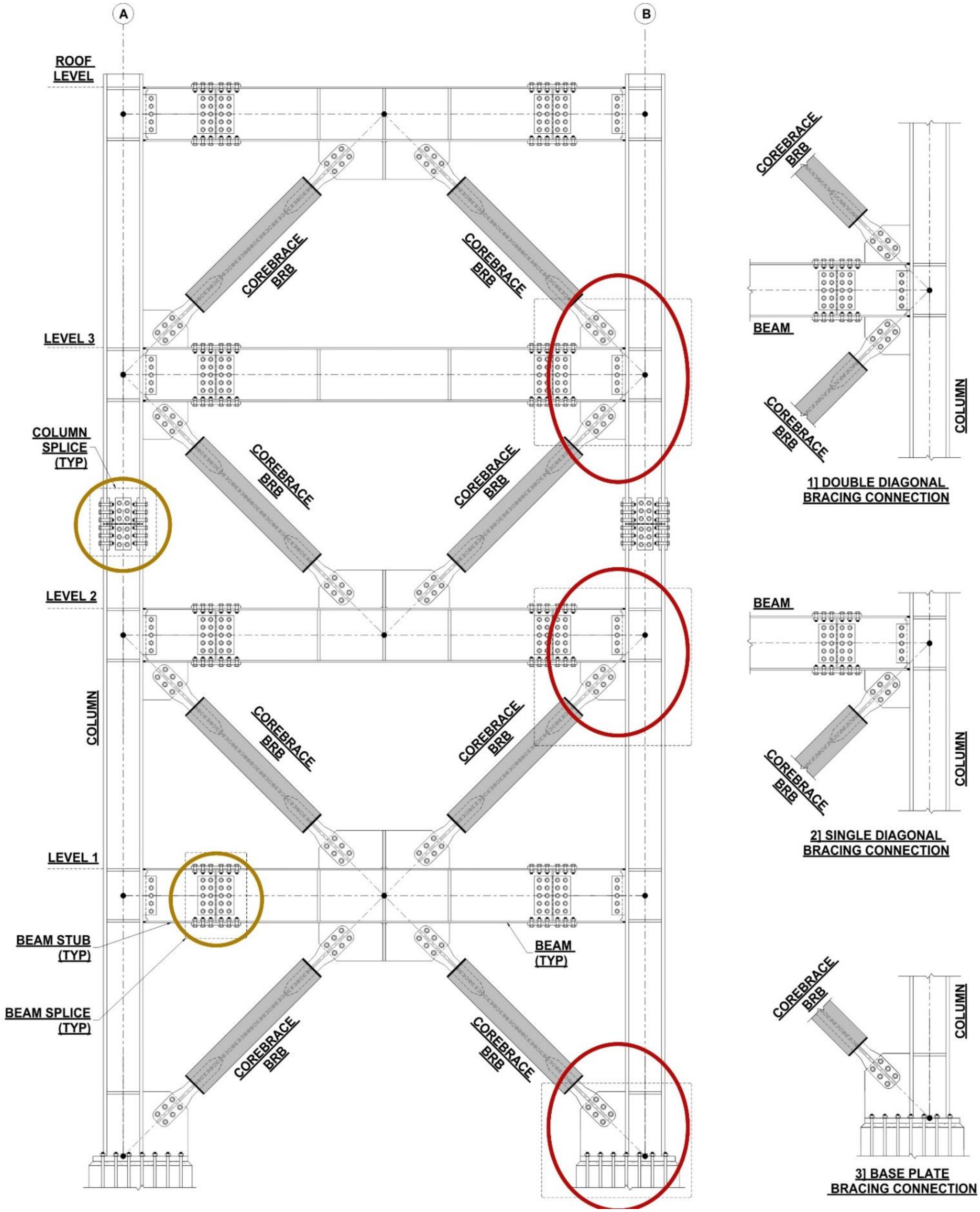


Figure 1 – Configurations of Corner and Other Connections in Buckling-Restrained Braced Frames (BRBFs)

Corner bracing configurations in Buckling-Restrained Braced Frames (BRBFs), as illustrated in Figure 1, are employed where braces terminate near structural corners in plan or elevation. These connections are crucial in seismic zones for effective axial force transfer while minimizing moment imbalances or instability in adjacent frames.

Three common corner BRB configurations include:

1. Base Plate Bracing BRB Connection
2. Single Diagonal Corner Bracing BRB Connection
3. Double Diagonal Corner Bracing BRB Connection

Each system presents unique challenges related to force transfer, seismic detailing, and integration with surrounding members.

1. Base Plate Bracing BRB Connection

This connection provides the critical interface between the BRB and the foundation through gusset plate connection. Components include the bolted BRB end fitting, gusset plate, steel base plate, and anchor bolts. Additional elements like shear keys or lugs may be incorporated to resist out-of-plane loads.

Design Requirements:

- Axial Load Transfer must consider expected brace strengths as per AISC 341-16:

$$P_{tension} = \omega R_y P_{ysc}$$

$$P_{compression} = \beta \omega R_y P_{ysc}$$

- Base Plate & Anchor Design must meet AISC 360-16 and ACI 318-19 (Chapter 17).
- Welds should conform to AWS D1.8 for seismic performance.
- Gusset Plates must provide flexibility for story drift and brace-end rotation.

Robust anchorage and flexible gusset detailing are crucial to avoiding brittle behavior and ensuring ductile performance at the base.

2. Single Diagonal Corner Bracing BRB Connection

This configuration features a single diagonal BRB extending across a bay between two levels. It is commonly used in upper or mid-story frames for lateral force resistance. However, the inclined layout introduces unbalanced vertical forces on beams, requiring additional attention.

Key Features:

- Diagonal BRB connects across floors using gusset plates.
- Beam-column joints experience additional vertical forces from brace action.

Design Considerations:

- Use the same axial load amplification as per AISC 341-16:

$$P_{tension} = \omega R_y P_{ysc}$$

$$P_{compression} = \beta\omega R_y P_{ysc}$$

- Reinforce beams for axial and vertical components from the BRB.
- Gussets must allow for out-of-plane deformation and brace rotation.
- Bolts and welds must remain elastic under seismic loads per AISC 360 and AWS D1.8.

This layout is efficient but introduces non-negligible vertical reactions that must be properly addressed through detailing and capacity checks.

3. Double Diagonal Corner Bracing BRB Connection

This configuration incorporates two opposing BRBs in the same bay, typically converging at the center beam at corner locations. It offers balanced axial force transfer and symmetrical seismic resistance.

Advantages:

- Vertical components of axial forces counteract each other.
- Improved frame symmetry and energy dissipation capacity.

Design Challenges:

- Complex gusset layout and coordination of brace geometry.
- Central beam segment may require additional reinforcement, such as doubler plates to carry combined axial forces.
- Detailing must address potential clashes at architectural corners.

Code-Based Requirements:

- Design forces follow the same capacity principles.
- All components (gussets, bolts, welds) must meet AISC 341, AISC 360, and AWS D1.8 standards.

This system is ideal for perimeter or zones subject to geometric constraints and high seismic demand

Column Splices in BRBF Systems

Column splices ensure vertical force continuity and must provide ductility under seismic loading.

AISC 341-16 Requirements:

- Splice strength ≥ 50% of column expected strength: $0.5R_yF_yA_g$
- Location: 18–36 inches above the beam-column joint to reduce moment demand.

Splice alignment, tolerances, and detailing for ductility are essential for seismic performance.

Beam-to-Beam Stub Splices

Stub splices are used to facilitate gusset plate installation, allow for BRB pre-installation, and provide clearance from plastic hinge regions.

Design Notes:

- Use bolted flange and web splice plates as required.
- Ensure elastic performance under seismic loads.
- Design as slip-critical per AISC 341 and RCSC.

Properly placed stub splices greatly enhance constructability, repair access, and the overall resilience of the BRBF system.

2. Analysis and Design methods for Corner connections in BRBF

The analysis and design of corner connections in Buckling-Restrained Braced Frames (BRBFs) require a combination of linear elastic, nonlinear static (pushover), and dynamic response history analyses, depending on the seismic design category and performance objectives. Linear elastic analysis is typically used for preliminary member sizing and force distribution. However, due to the inelastic nature of BRBs, capacity design principles must be applied—ensuring that all non-brace components (e.g., beams, columns, gusset plates, welds, bolts, and splices) remain essentially elastic while the BRBs yield. The AISC 341-16 provisions govern design strength requirements using amplified axial demands: $P_{tension} = \omega R_y P_{ysc}$ and $P_{compression} = \beta \omega R_y P_{ysc}$ where ω accounts for strain hardening, R_y for material overstrength, and β for compression adjustment.

In Buckling-Restrained Braced Frames (BRBFs), each brace includes a steel yielding core enclosed in a concrete or steel casing to prevent buckling. Under seismic loading, these braces yield in both tension and compression, while the surrounding casing ensures stability. The beam at the brace intersection point experiences significant vertical forces due to unbalanced brace actions and must be carefully designed to resist these loads without excessive deformation or yielding.

The UFM, as outlined in the AISC Steel Construction Manual, is a code-supported analytical technique used to resolve the axial force in a brace into orthogonal components—typically horizontal and vertical. These components are assumed to be distributed uniformly across the gusset interfaces with adjoining beams and columns. This method is frequently applied in the design of bolts and welds, selection of gusset plate thickness, and general connection detailing. Despite its widespread acceptance, the UFM is limited by its reliance on linear-elastic behavior. It does not account for inelastic deformation, localized yielding, stress redistribution, or post-yield behavior, and it lacks the capacity to simulate potential failure modes such as gusset plate buckling.

The KISS Method is a simplified, heuristic-based approach often used during the conceptual or preliminary design phase. It assumes a 60-degree load spread from the brace into the gusset plate, resulting in a triangular stress block for estimating required gusset geometry. Although highly practical for quick sizing decisions, this method is not formally codified in AISC provisions. It does not include bolt or weld design and does not address complex behaviors such as out-of-plane deformation, bolt group eccentricity, or interaction with surrounding framing members. As such, the KISS Method is most appropriate for initial design studies and must be supplemented by more robust analytical tools such as UFM or Component-Based Finite Element Modeling (e.g., using IDEA StatiCa Connection) during final design verification.

The analysis and design of corner connections in Buckling-Restrained Braced Frames (BRBFs) require a multi-level approach that combines linear elastic, nonlinear static (pushover), and dynamic response history analyses, depending on the seismic design category and performance objectives. While linear elastic analysis is typically sufficient for preliminary member sizing and force distribution, the inelastic behavior of BRBs under seismic loading necessitates the application of capacity design principles. These principles ensure that non-yielding components—such as beams, columns, gusset plates, welds, bolts, and splices—remain essentially elastic while the BRB elements undergo yielding.

Design strength requirements for BRB axial forces are governed by AISC 341-16, which prescribes amplified demands to reflect potential overstrength and inelastic response:

- Tension capacity: $P_{tension} = \omega R_y P_{ysc}$
- Compression capacity: $P_{compression} = \beta \omega R_y P_{ysc}$

Here, ω accounts for strain hardening, R_y is the material overstrength factor, and β is a compression adjustment factor for BRBs.

Each BRB consists of a yielding steel core encased in a buckling-restraint mechanism—typically a concrete-filled steel casing or steel tube—which prevents global buckling. This configuration enables the brace to undergo controlled inelastic deformation in both loading directions, providing stable hysteretic energy dissipation during seismic events.

2.a. Analytical Methods for BRBF Corner Connections

1. Uniform Force Method (UFM)

The Uniform Force Method (UFM), as outlined in the *AISC Steel Construction Manual* (AISC 360, 2016), is widely used for the analytical design of BRB connections. It resolves the brace's axial force into orthogonal components—typically horizontal and vertical—and distributes them uniformly across gusset plate interfaces with adjacent beams and columns. UFM is typically employed in:

- Bolt and weld design,
- Gusset plate thickness selection, and
- General connection detailing.

However, UFM is limited to linear-elastic analysis and does not capture:

- Inelastic strain distribution,
- Stress redistribution,
- Local yielding, or
- Buckling of gusset plates or other components.

Thus, while UFM is practical for code-based elastic design, it lacks the fidelity needed for performance-based seismic design or post-yield evaluation.

2. KISS Method

The KISS (Keep it Simple, Stupid) Method is a heuristic, geometry-based design aid commonly used in preliminary design phases. It assumes a 60° load dispersion from the BRB into the gusset, creating a triangular stress block to estimate gusset plate geometry and size. While this approach is:

- Quick and practical for conceptual design,
- Useful for early-stage feasibility checks,

This method is not codified in AISC, and has several limitations:

- Does not address bolt or weld design,
- Ignores out-of-plane deformation and eccentricity,
- Does not model interaction with surrounding members or brace flexibility.

Therefore, the KISS Method should be limited to preliminary sizing and must be followed by a more rigorous evaluation using validated methods such as UFM and CBFEM.

3. Component-Based Finite Element Modeling (CBFEM)

For final design and verification, especially in non-standard corner BRB connections, the use of Component-Based Finite Element Modeling (CBFEM), such as through IDEA StatiCa Connection, is recommended. CBFEM integrates the component method (based on design codes) with nonlinear finite element analysis, offering:

- Local modeling of yielding and plastic hinges,

- Bolt and weld simulation using realistic force-deformation behavior,
- Stress/strain visualization and check of von Mises stresses, plastic strains, and strain concentrations.

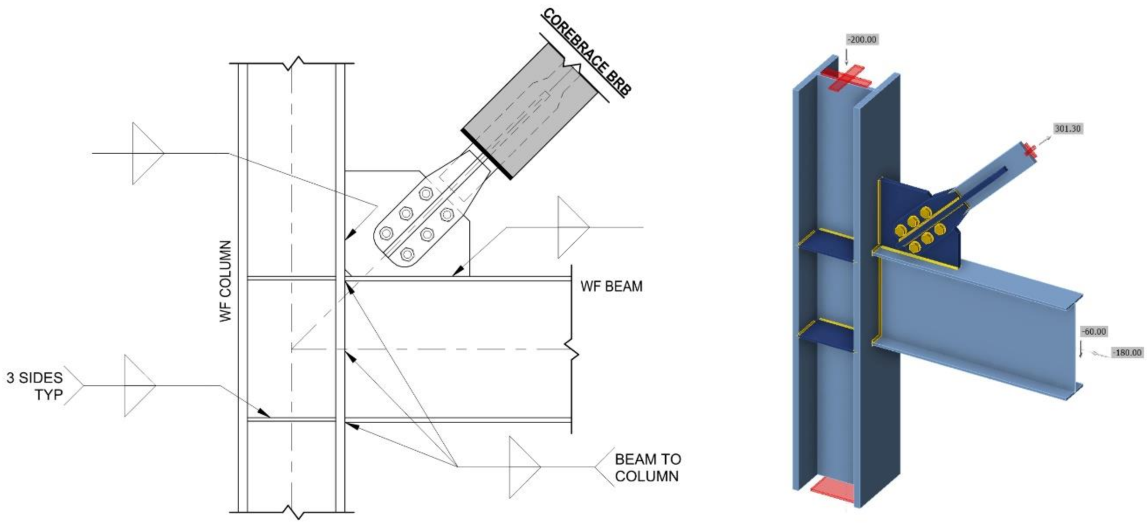
Overall, CBFEM enables us to evaluate actual joint behavior, particularly in geometrically complex or seismically critical regions, making it a vital tool for final-stage verification and detailing of BRBF corner connections. Table 1 compares Component Method (CM), Finite Analysis (FEA) and Component Based Finite Element Method (CBFEM) on various parameters.

Table 1 – Comparison of Component Method, Finite Element Analysis and Component Based Finite Element Method

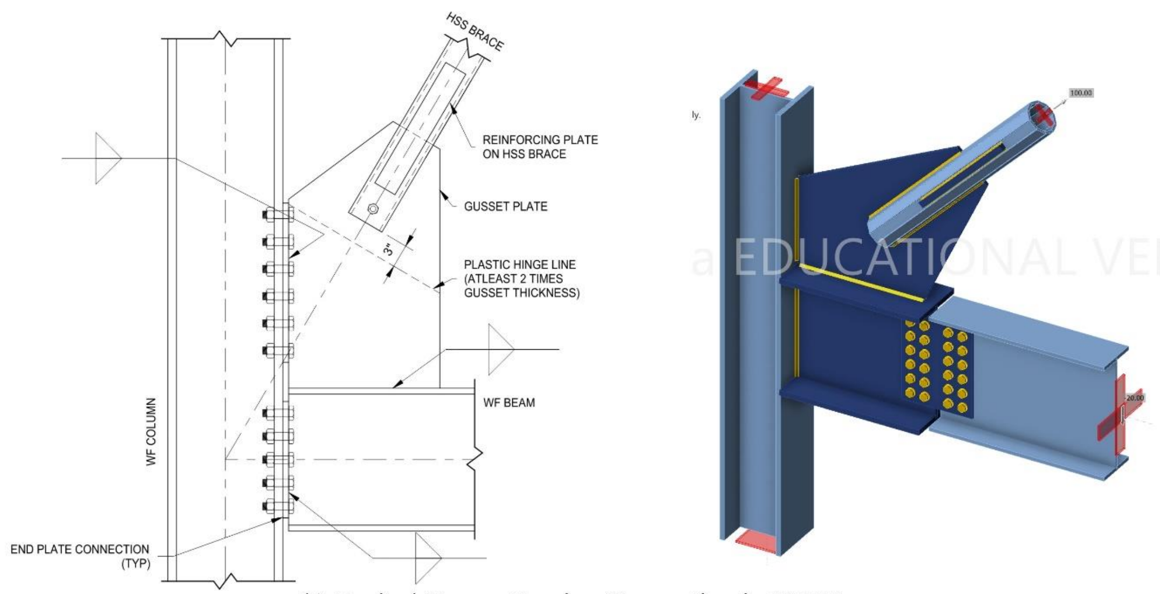
Comparison of Component Method (CM), Finite Element Analysis (FEA) and Component Based Finite Element Method (CBFEM)			
Parameter for Comparison	**Component Method**	**Finite Element Analysis**	**Component Based Finite Element Method**
1. Definition	Analytical method based on simplified mechanical models of components (eg. Bolts, welds, plates).	Numerical method with full discretization of the joint or structure using shell or solid elements.	Hybrid approach combining component method and finite element analysis.
2. Inelastic Behavior	Not modelled explicitly, relies on empirical reduction factors or conservative assumptions.	Full non-linear behavior can be modelled and evaluated.	Local plasticity and yielding modelled in joint (typically limited to 5% strain limit).
3. Bolt and Weld Behavior	Analytical design formulas based on the codal provisions.	Requires non-linear contact definitions, detailed meshing and post-processing.	Modelled as non-linear components with behavior governed by code - based formulations.
4. Code Compliance Integration	To be performed manually – Based on engineers judgement and experience.		Built-in code checks (eg. AISC, EN, CSA) with automatic verification reports.
5. Computational Time	Very Low – Hand calculations or spreadsheets.	High- hours or days for complex joints, especially under cyclic or dynamic loading.	Moderate – typically a few minutes per joint.
6. Skill Level Required	Basic – Suitable for practical design engineers.	Advanced – Requires specialized knowledge.	Intermediate – Design engineers with minimal training.
7. Use Cases	Standard , repetitive connection types (e.g., end-plate, bolted angles).	• Non-Standard Joints. • Fracture Studies. • Experimental Calibration. • Performance-based Seismic Design etc.	Complex and non-standard steel joints requiring realistic behavior evaluation.
8. Example of Tools	• Spreadsheets. • AISC Design Examples. • Manual Calculations.	• ABAQUS • ANSYS • SAP 2000 CSI.	• IDEA StatiCa Connection • IDEA StatiCa Member • Hilti PROFIS
9. Output Type	• Internal Forces. • Demand-to-Capacity Ratios (DCRs). • Simple Stiffness Metrics	• Full Stress Strain Displacement Contours. • Reaction Forces. • Time History results	• Overall Check • Strain Check • Check of Plastic Strains and von Mises Stress etc.
10. Best used for	Preliminary or repetitive design of standard connections.	Detailed research studies and advanced evaluation of connections.	Verification and validation of all types of steel joints.

3. Comparison of Corner Bracing Connections in SCBF and EBF

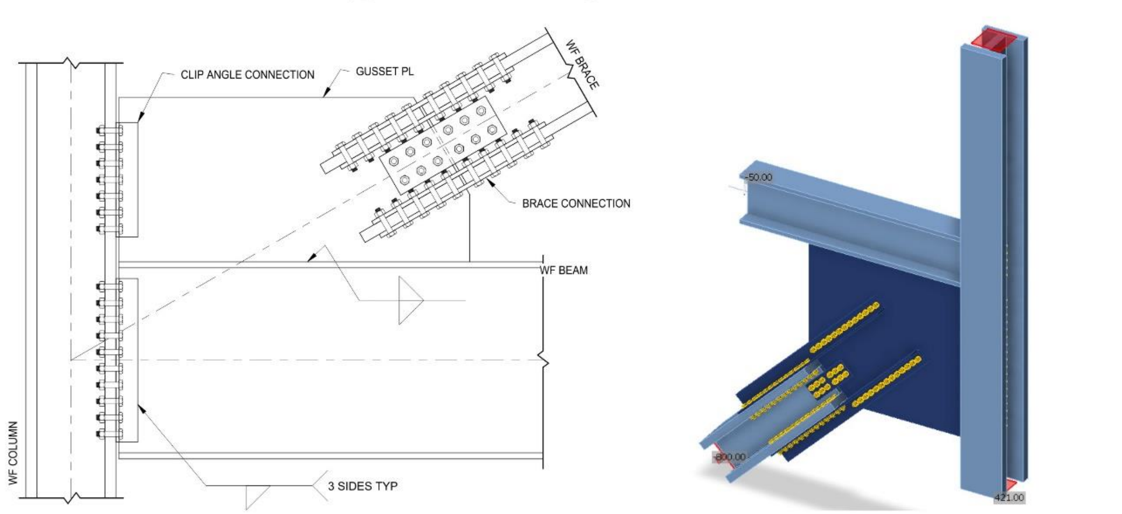
Corner bracing connections in Buckling-Restrained Braced Frames (BRBFs), Special Concentrically Braced Frames (SCBFs), and Eccentrically Braced Frames (EBFs) exhibit distinct structural behaviors due to differences in their load path mechanisms, energy dissipation strategies, and brace-to-frame interaction. These differences have significant implications on connection detailing, constructability, and seismic performance.



a) Typical Corner Bracing Connection in BRBF



b) Typical Corner Bracing Connection in SCBF



c) Typical Corner Bracing Connection in EBF

Figure 2 – Comparison of Corner Bracing Connection in BRBF, SCBF, EBF and OCBF

In BRBFs, the braces as illustrated in Figure 2.a) consist of a steel yielding core encased in a buckling-restraining mechanism that prevents lateral deformation, allowing the core to yield in both tension and compression. This feature provides stable, repeatable hysteresis loops even under large cyclic drifts. At structural corners—typically at the periphery of the plan or elevation—BRBF corner bracing connections must address several challenges. Firstly, when single diagonal braces are used, an unbalanced vertical force component is introduced into the beam-column joint, which must be resisted by axial and flexural strength of the surrounding members. Secondly, gusset plates connecting BRBs at corners must be designed with sufficient out-of-plane flexibility to accommodate expected frame drift and brace-end rotations. Detailing becomes more intricate at corners due to the geometric limitations in routing BRBs and maintaining adequate brace clearances from architectural boundaries. Additionally, BRBF corner connections must comply with AISC 341-16 clause F4 provisions, including expected strength design using strain-hardening and overstrength factors ($\omega R_y P_{ysc}$ in tension, $\beta\omega R_y P_{ysc}$ in compression), as well as AWS D1.8 requirements for weld toughness, and RCSC criteria for slip-critical bolting. While fabrication and detailing are relatively complex, BRBF corner connections offer high ductility, robustness, and reduced risk of strength degradation during seismic events.

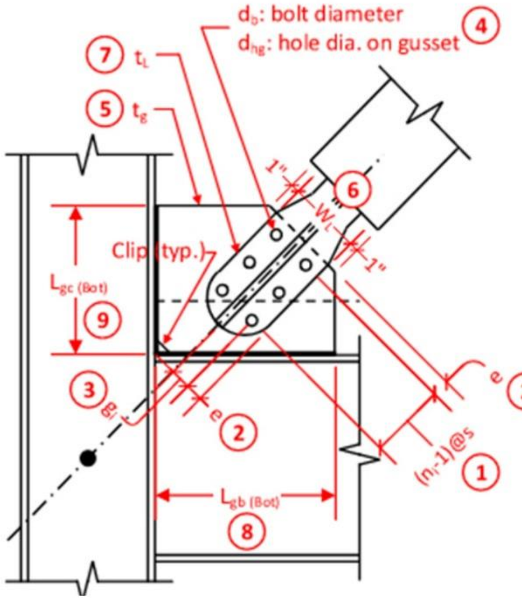
In SCBFs, the corner bracing connections as illustrated in Figure 2.b) utilize conventional steel braces, which are designed to yield in tension and are allowed to buckle in compression. While this system is relatively simple to detail, seismic performance is less reliable due to the inherently asymmetric hysteretic behavior and degradation of compressive capacity after brace buckling. At corners, SCBF systems often use X- or chevron-type configurations, but where a single diagonal brace is used, the system introduces significant unbalanced vertical forces similar to BRBFs. Gusset plates in SCBFs must be designed to allow brace buckling rotation and frame deformation while preventing premature local buckling. The connection must also ensure that columns and beams are capable of sustaining increased axial and bending demands resulting from brace buckling and the redistribution of forces. SCBFs are governed by AISC 341-16 Clause F2, which emphasizes capacity design principles to protect the connection region and ensure that yielding and buckling occur within the braces rather than in the surrounding frame. While SCBF corner connections are easier to analyze and construct than BRBFs, their energy dissipation capacity is limited, and degradation under cyclic loading leads to lower cumulative ductility and potential for soft-story behavior.

In contrast, EBFs transfer seismic energy into a deliberately designed link beam rather than allowing the brace to yield or buckle. In EBF corner connections as illustrated in Figure 2.c), braces are offset such that they frame into the beam at a specific eccentricity, creating a short link segment that is designed to yield in either shear (short links) or moment (longer links). The energy dissipation is concentrated within this link, while braces remain elastic and act as force-delivery mechanisms. This shift in the energy dissipation mechanism reduces demands on gusset plates and braces themselves but substantially increases the design complexity at the link and surrounding framing. At corners, designing EBF connections becomes particularly challenging due to alignment constraints, torsional behavior, and the difficulty of placing links without interfering with floor systems or architectural openings. The link beam must be carefully detailed with adequate length, flange and web stiffeners, and plastic hinge zones to allow predictable and stable yielding. Per AISC 341-16 clause F3, link design requires verification of shear and flexural strength, rotational ductility, and deformation capacity under cyclic loading. Additionally, braces must be detailed to ensure proper axial load transfer into the link ends without inducing local distortion. While EBF corner connections provide excellent energy dissipation and exhibit minimal strength degradation, the system demands high precision in detailing, fabrication, and inspection, especially when links and braces intersect at unconventional angles or positions.

Overall, BRBF corner bracing connections provide high ductility and energy dissipation, but require careful detailing of gusset plates and member interfaces. SCBF corner bracing connections are simpler and more conventional, but are limited by brace buckling and reduced hysteretic stability. EBF corner bracing connections, although extremely effective in isolating and controlling energy dissipation, involve complex link detailing and geometric challenges, particularly at building perimeters.

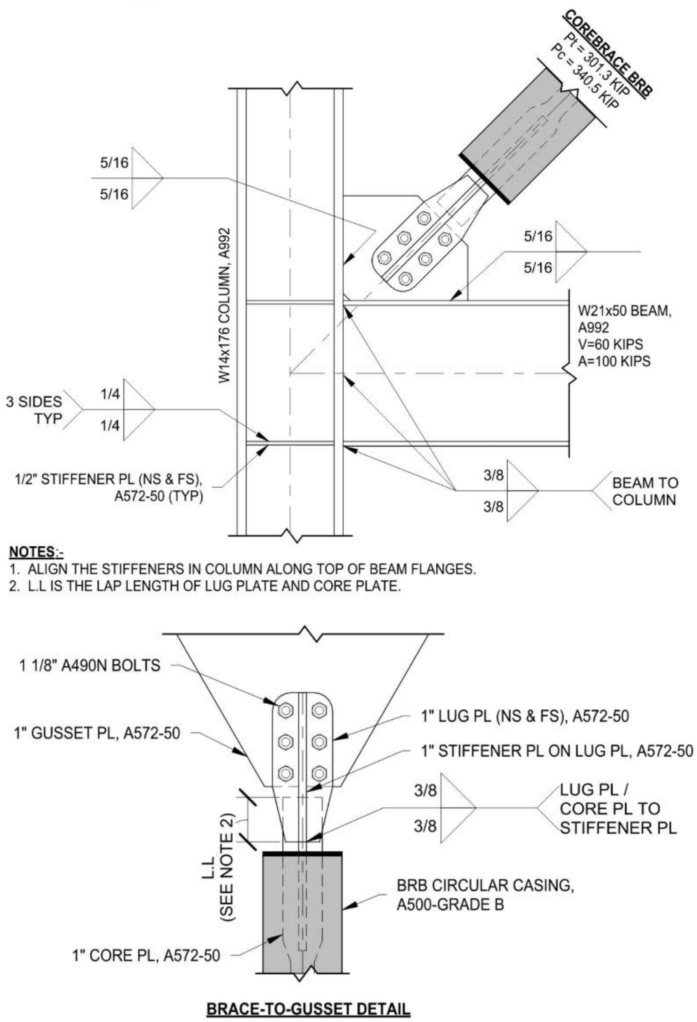
4. Problem Description

The objective of this example is to verify the Component-Based Finite Element Method (CBFEM) for a Corner Single-cored bolted brace-to-gusset connection located at the fourth story of a Buckling-Restrained Braced Frame (BRBF) in a steel building. The design details of the considered connection are as shown in Figure 3. The results obtained using calculation methods (UFM) based on the *Steel Construction Manual*, Fourteenth Edition (AISC 360, 2016), and the *Seismic Design Manual*, Third Edition (AISC 341, 2016), are then compared with those obtained from CBFEM using capacity design analysis in IDEA StatiCa software, version 25.

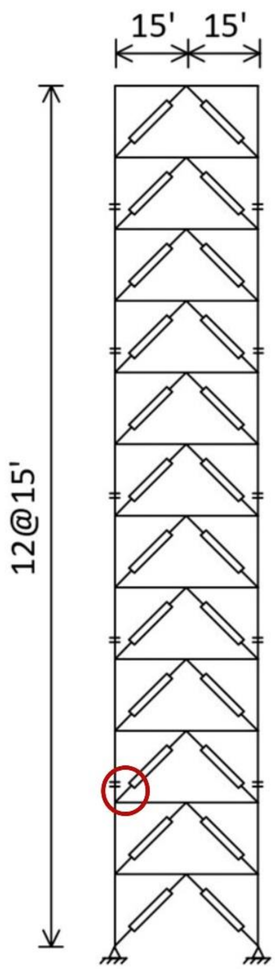


a) General Sketch for Corner Type BRB Connection

b) Image of Constructed Corner BRB Connection



c) Connection Design Details as per AISC 360-16 and 341-16



d) Building Elevation and Location of Considered Connection

x

Figure 3 – Bolted-Type Corner CoreBrace Connection Details

Figure 3 presents a comprehensive overview of a Corner Type Buckling-Restrained Brace (BRB) Connection using sketches, photos, detailing, and elevation context. Here's a detailed explanation of each subfigure:

a) General Sketch for Corner Type BRB Connection

This annotated diagram identifies key geometric parameters and components of a corner BRB connection:

- Bolt geometry and spacing (d_b, d_{hg}, s, e) are shown clearly.
- Thicknesses (t_c for core plate, t_g for gusset plate) and lap lengths (L_{bp}, L_{gc}) are identified.
- Clip angles and lug-core assembly orientation are indicated for reference.
- The load transfer path is clearly represented, guiding the expected axial force flow from brace to gusset to the beam-column joint.

b) Image of Constructed Corner BRB Connection

This real-life image obtained from CoreBrace website shows the actual fabricated assembly of a corner BRB connection:

- The brace frames into the weak axis of the column, a common configuration in seismic design when architectural constraints or framing geometry dictate such alignment.
- The wide gusset plate connects the brace to both the beam and the column.
- Multiple bolts transfer axial forces through lug plates into the gusset, and welds ensuring robust plate connectivity.

c) Connection Design Details (per AISC 360-16 & 341-16)

The details for the member as well as connections obtained from CoreBrace connection schedule are as presented below,

Member Details

1. Beam cross-section
- W21x50, ASTM A992
2. BRB Core
- BRB A_{sc} as 5 in.^{2}
- Casing type is square
- Fy_{sc} specified yield stress (Fy_{sc}) range of core plate = 38ksi to 46ksi
- L_b = Length of CoreBrace tip to tip = 216 6/16 in.
- L_c = Length of Casing = 178.63 in.

Connection Details

1. Connection Type
   Single-cored bolted brace-to-gusset connection.
2. Welds (Fillet welds at all applicable locations)
- E70xx electrode
- 3/8" double sided fillet weld between stiffener plate and lug plate.
- 3/8" double sided fillet weld between beam and column.
- 5/16" double sided fillet weld between gusset plate and top flange of beam.

- c = Core extension length out of casing = 2.25 in.
- LL_c = Length of Lug within Casing (incl xL) = 8.27 in.
- $\beta = 1.13$
- $\omega = 1.31$
- $R_y = 1$

Plate Details

1. **Gusset Plate**

- PL 1”, ASTM A572-50
- LL_g = Length of Lap on Gusset = 12.38in
- a = Gap between core and gusset = 4in

2. **Stiffener Plates in column**

- PL ½”, ASTM A572-50

3. **Core Plate**

- PL 1”, ASTM A572-50

4. **Lug Plate**

- PL 1” , ASTM A572-50
- W_L = Width of Lug = 7.5in
- W_1 = Width at Reduced Section of Lug = 5.02in
- LSL = Total Length of Lug minus Transition = 23.13 in.
- 5/16” double sided fillet weld between gusset plate and column flange.
- 1/4” double sided fillet weld – all three sides between stiffener plate and connecting plate or member.

3. Bolts

- 1 1/8” A490-N type bolts connecting lug plate to gusset plates.
- n_i = Number of bolts in inner row = 3
- n_o = Number of bolts in outer row = 0
- g = Gauge between outer and inner bolt rows = 2 3/16 in.
- s = Bolt spacing = 4 in.
- e = Typical bolt edge distance = 1.63 in.
- b_r = Distance to start of radius from first outermost bolt = 1 in.

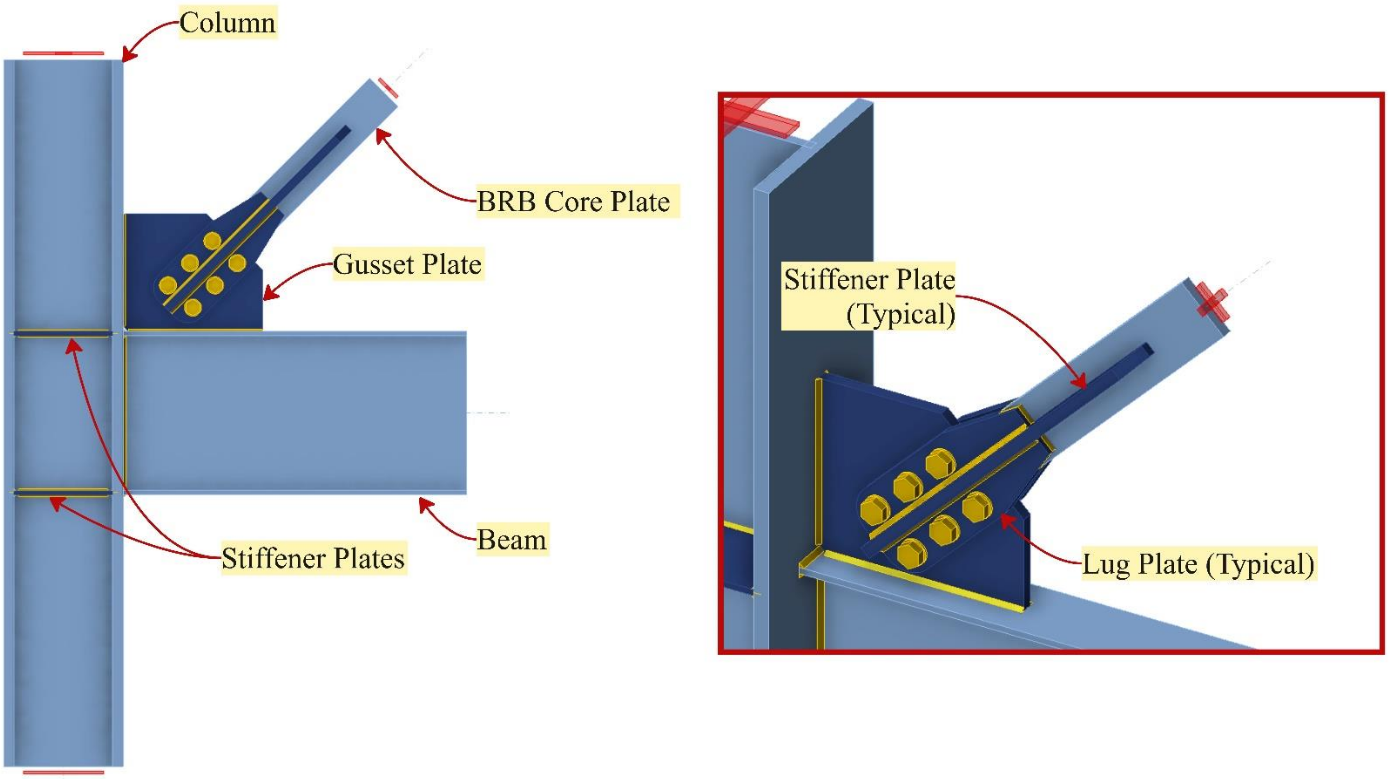
d) Building Elevation and Connection Location

The diagram Figure 3.d) shows the overall corner BRBF configuration in elevation for considered connection:

- 12 stories are spaced at 15 ft intervals, with braces arranged in a Chevron pattern.
- The red circle highlights the specific corner connection under study in this project.
- This context is essential for understanding the gravity and lateral load path, and where the selected connection fits within the broader seismic force-resisting system (SFRS).

5. Modelling and Analysis of connection in IDEA StatiCa

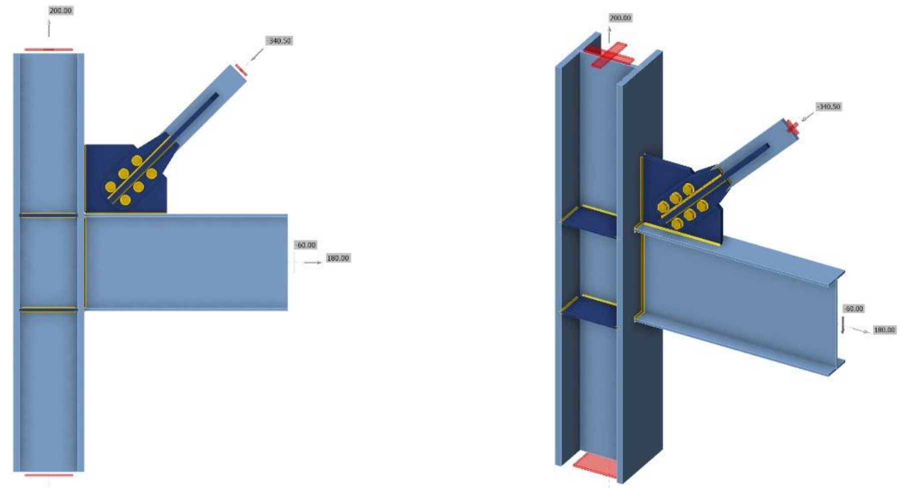
The connection modeling in IDEA StatiCa begins with defining the members—specifically, the beam, column and core plate. Next, the gusset plates, stiffener plates and lug plates are defined and arranged according to the connection details shown in Figure 3. The core plate is connected to the gusset plates through the lug plate using bolted connections. The stiffener plates are connected to the lug plates by fillet welds applied in the shop, as illustrated in Figure 4.



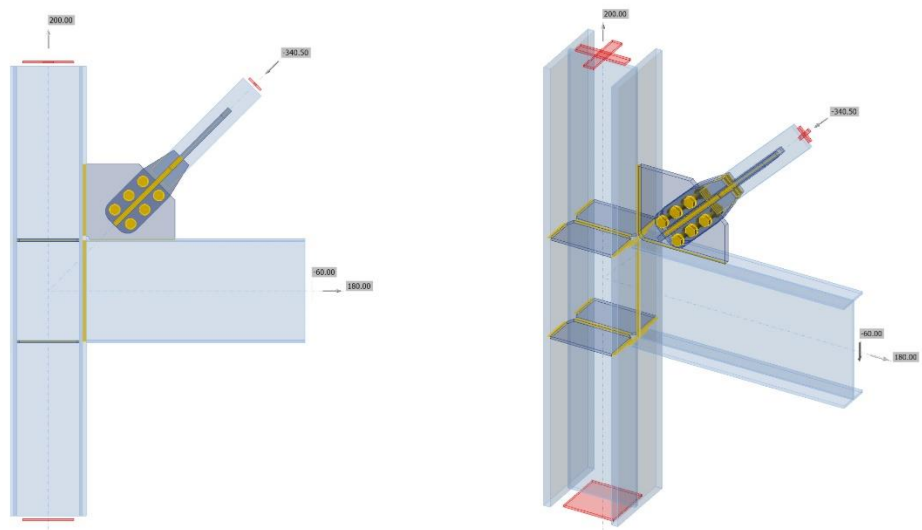
a) Defination of elements in base model for bolted CoreBrace BRB Corner Connection – CBFEM

Figure 4 – Base Model of the Corner Bolted-Type CoreBrace BRB in CBFEM

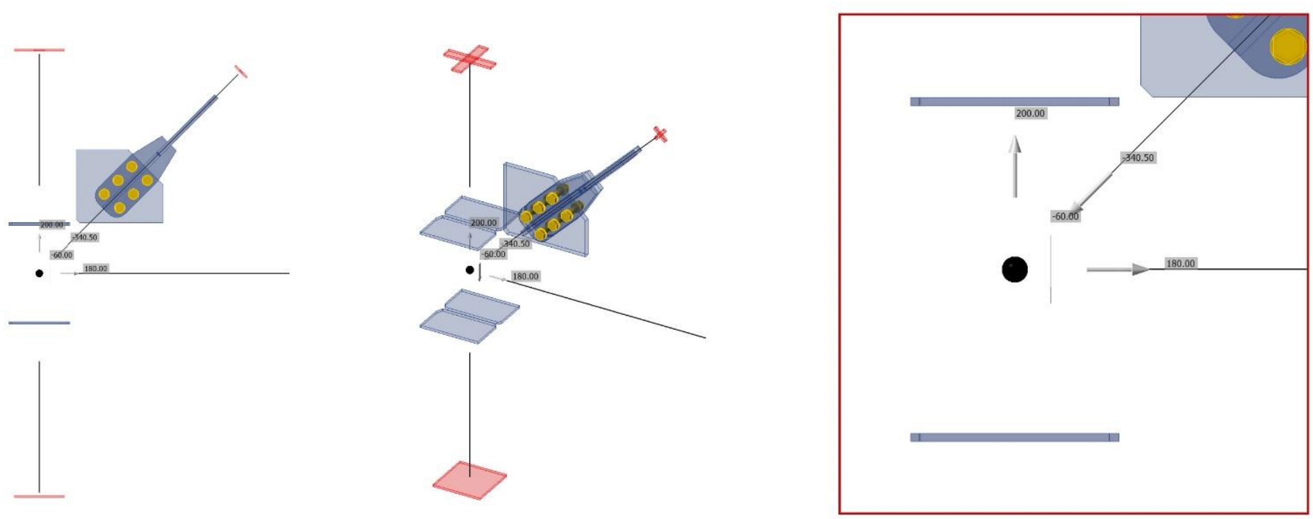
The column is assigned as a bearing member in the CBFEM model. A model type of N-Vy-Vz-Mx-My-Mz (Fixed) is assigned to the beam and column, while the bracing members are assigned a model type of N-Vy-Vz (Pinned). This setup ensures that forces are applied at the nodes, as shown in the wireframe view in Figure 5.



a) Solid view of CoreBrace BRB
Corner base model - CBFEM



b) Transparent view of CoreBrace BRB
Corner base model - CBFEM

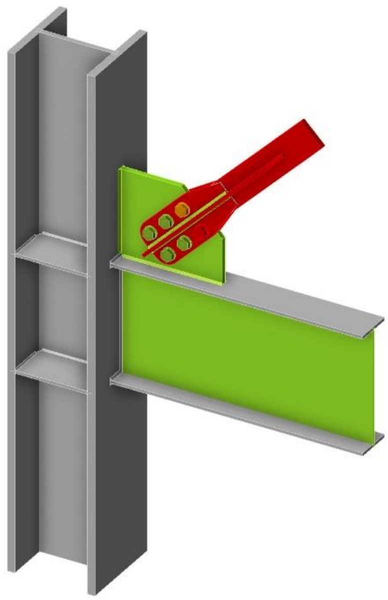


c) Wireframe view of CoreBrace BRB
Corner base model - CBFEM

Figure 5 – Connection Configurations in CBFEM: Solid, Transparent, and Wireframe Views

Visualization of connection or member failure can be easily predicted using the "traffic light" format of the overall check provided by the CBFEM after running the analysis. Additional details are shown in Figure 6.

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

Based on the observations for the given connection shown in Figure 6, the conclusions are summarized in Table 2 below.

Sr No	Color	Elements	Comments
1	GREY	Column, Stiffener plates in column etc.	Elements are safe for action of loads.
2	GREEN	Beam web, Gusset plate etc.	Elements are safe for action of loads.
3	ORANGE	Bolts etc.	Elements are on the verge of failure, for the applied loads.
4	RED	Lug Plate, Core Plate, Stiffeners plate etc.	Elements are failing for the applied loads; utilization is beyond 100%.

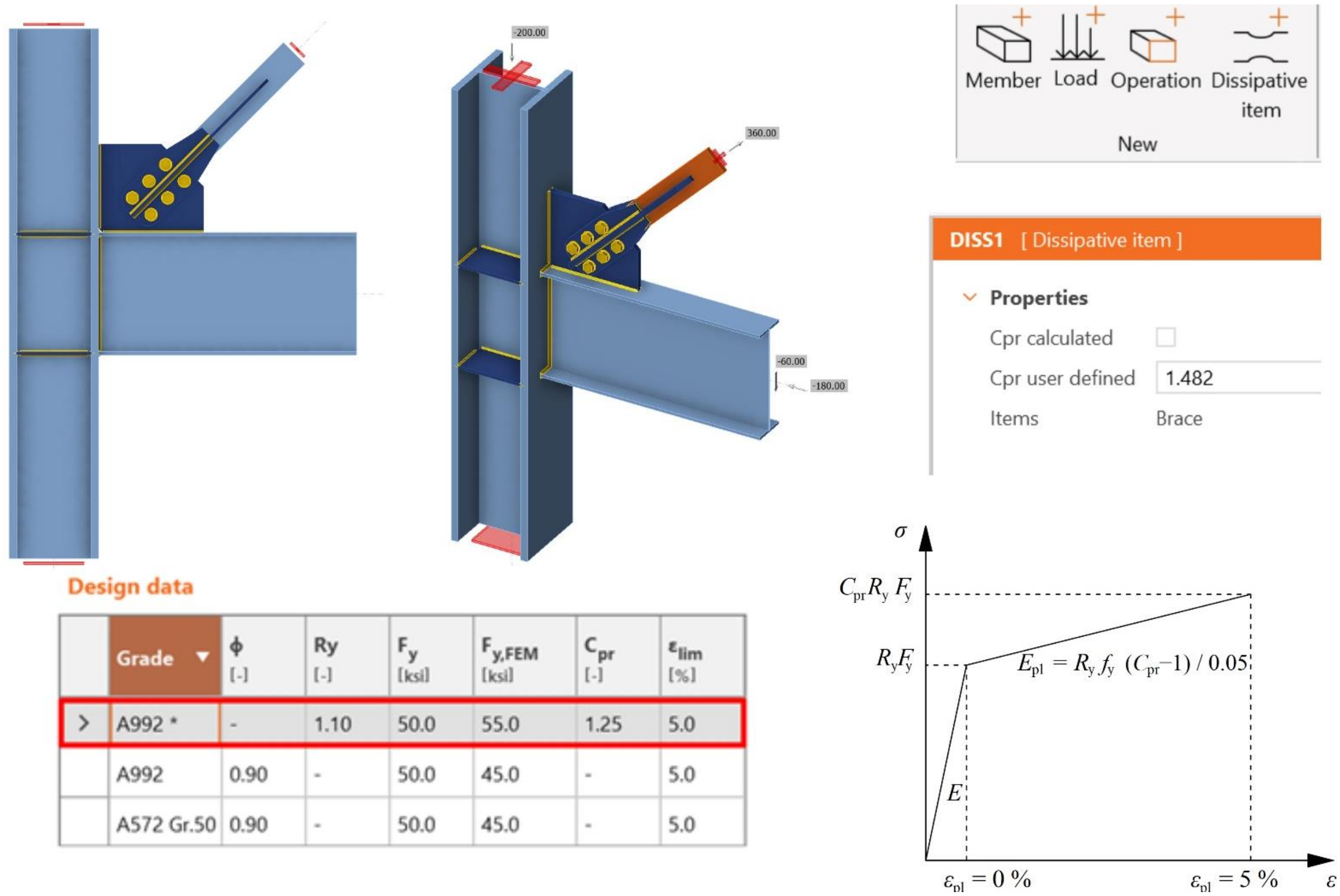
Table 2 – Traffic Light Visualization of the Corner BRB Connection in CBFEM

Capacity design analysis is performed for BRB connection in CBFEM. The core brace member is selected as dissipative items in this analysis as shown in Figure 7. The value for design factor (R_y) is considered in material properties by CBFEM, whereas the value for the adjustment factor (β *and* ω) is provided as a manual input in C_{pr} user defined value for the analysis considered.

where,

- β is compression strength adjustment factor
- ω is the stain hardening adjustment factor

- R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y



	Grade	ϕ [-]	Ry [-]	F_y [ksi]	$F_{y,FEM}$ [ksi]	C_{pr} [-]	ε_{lim} [%]
>	A992 *	-	1.10	50.0	55.0	1.25	5.0
	A992	0.90	-	50.0	45.0	-	5.0
	A572 Gr.50	0.90	-	50.0	45.0	-	5.0

Figure 7 – Capacity design analysis of BRB connection – CBFEM

6. Verification of Resistance – Capacity of Elements Subjected to Tension Load

The behavior of a corner BRB connection under seismic tension loads was investigated using the Component-Based Finite Element Method (CBFEM) in IDEA StatiCa. The capacity design analysis specifically focused on how the connection elements respond to increasing axial tension loads, applied incrementally from 300 kips to 500 kips. In the modeling setup, all brace members were defined as dissipative elements—that is, components expected to undergo inelastic deformation—while the remaining connection elements (e.g., gusset plates, bolts, stiffeners, and welds) were intended to stay elastic throughout the seismic event. This approach aligns with the capacity design philosophy recommended in AISC 341-16, which stipulates that the yielding should be confined to BRB cores, while other elements retain strength and stiffness to ensure reliable load transfer.

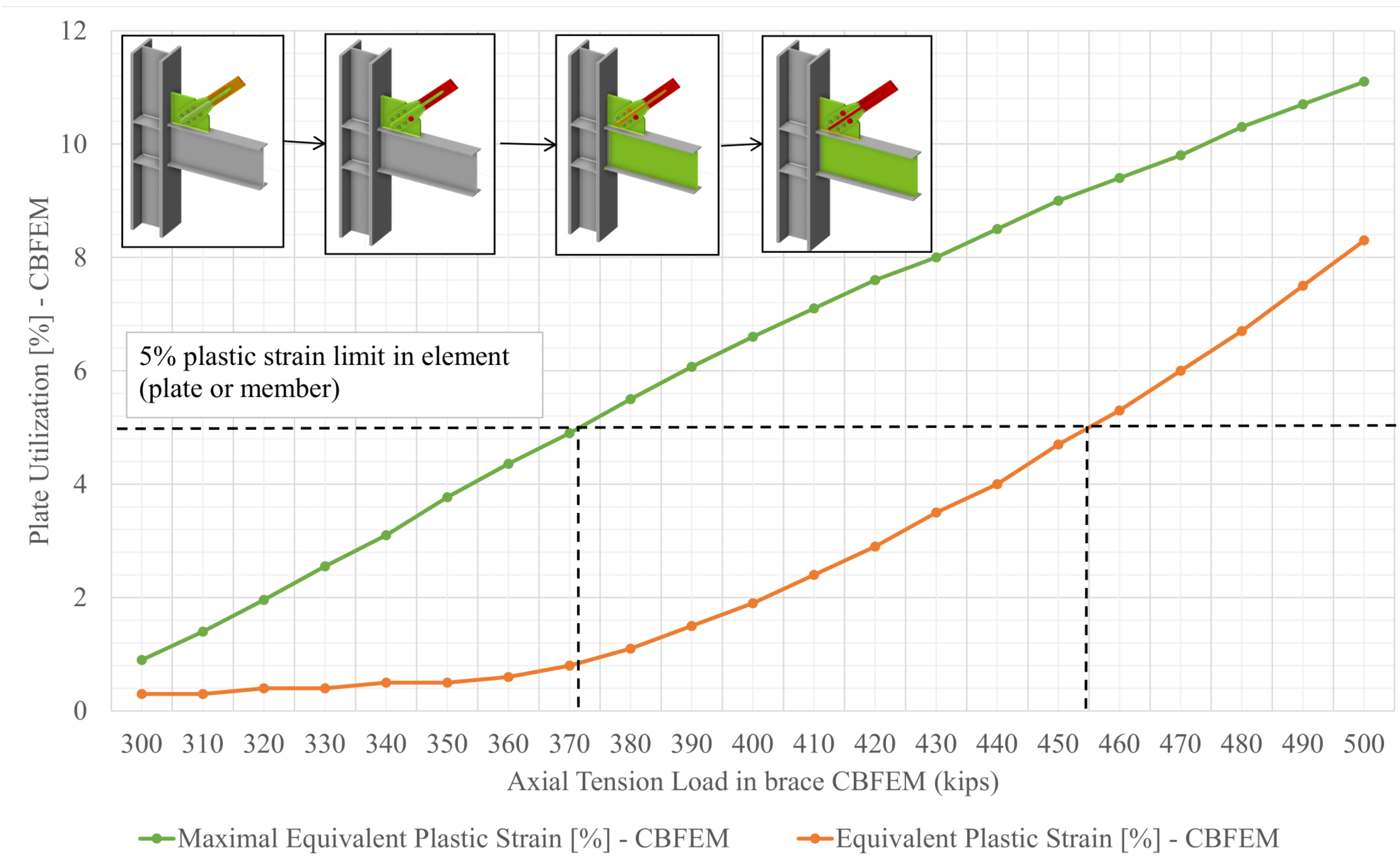
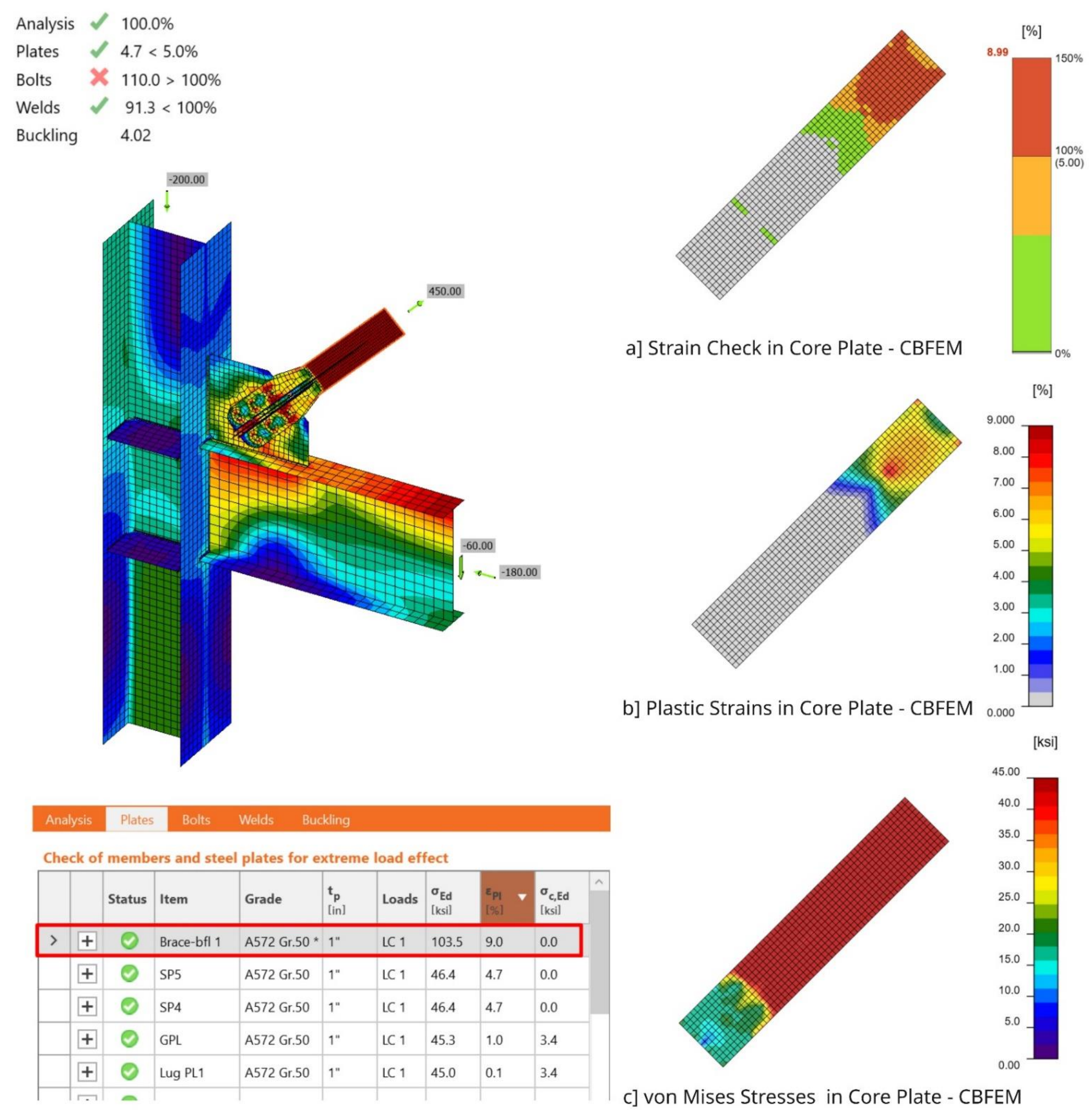


Figure 8 - Graph for Evaluation of Capacity of Elements for Tension Load in Brace– CBFEM

During the analysis, two important parameters were monitored as illustrated in Figure 8: equivalent plastic strain and maximal equivalent plastic strain. Equivalent plastic strain ($\varepsilon_{pl,eq}$) is a representative average of plastic deformation distributed across a particular connection element or component. It helps in evaluating the general extent of yielding and deformation in that element. On the other hand, maximal equivalent plastic strain ($\varepsilon_{pl,max}$) captures the highest value of localized plastic strain, typically found at critical points of stress concentration, such as bolt holes, weld toes, gusset corners, or points of geometric discontinuity. These two metrics provide complementary insights—while the former indicates overall ductility demand, the latter flags potential failure zones or hot spots.

As the axial tension was progressively increased in the BRB from 300 kips to 500 kips, the analysis revealed that maximal plastic strain values rose significantly faster than the equivalent strain values. Around 370 kips, the maximal strain crossed the 5% threshold, a commonly accepted upper limit for non-dissipative components in seismic design per IDEA StatiCa and AISC guidance. However, at that point, the equivalent plastic strain was still below 3%, indicating that the inelasticity was localized and not widespread across the gusset plate or weld zone. This result is considered acceptable, as localized yielding is expected near sharp transitions or connection details, and it does not compromise the global performance of the connection. As loading progressed beyond 400 kips, both strain values increased, but the discrepancy between them persisted—demonstrating that the yielding continued to be concentrated in specific areas while most of the connection remained within elastic limits.

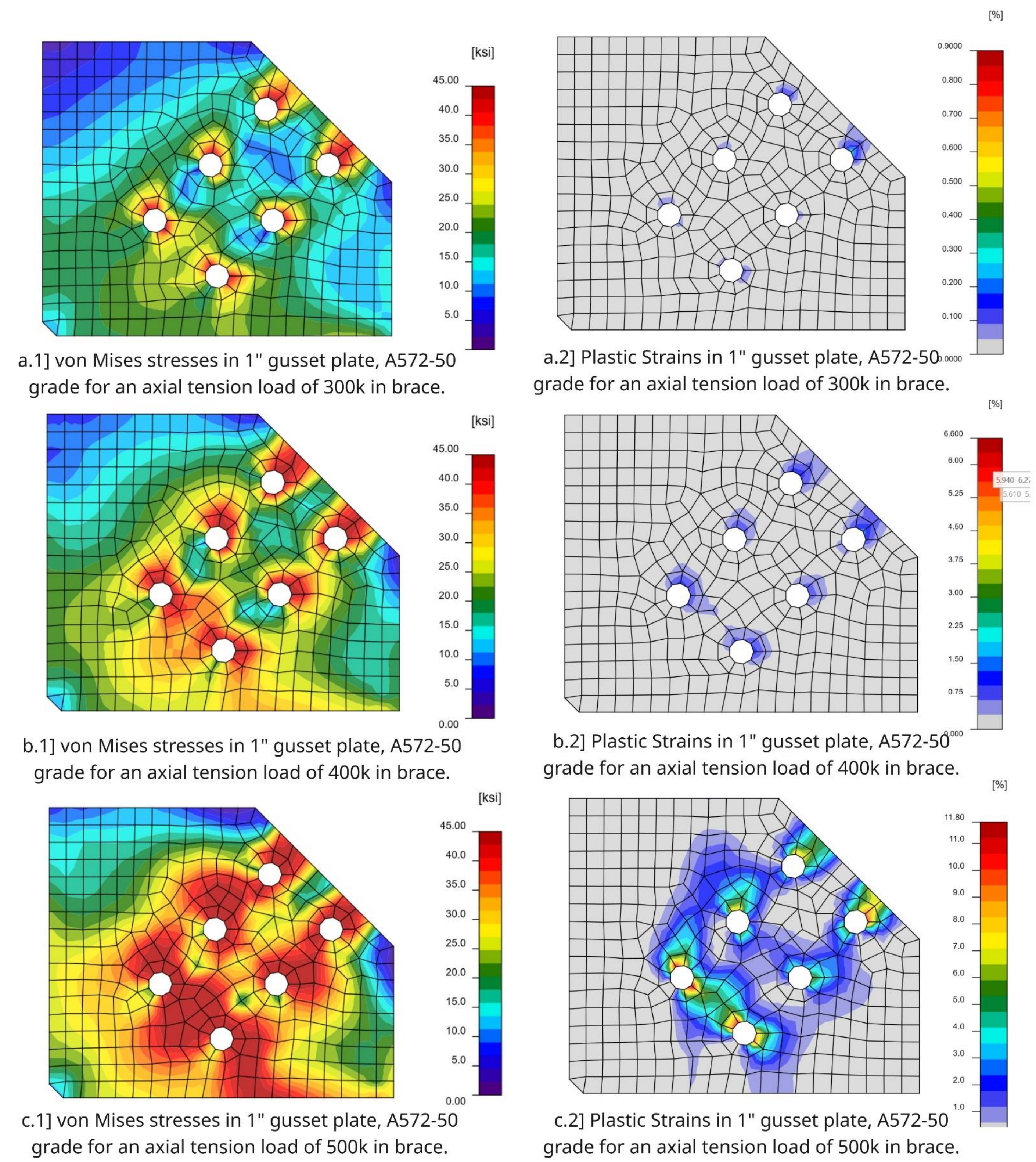


Status	Item	Grade	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]
✓	Brace-bfl 1	A572 Gr.50 *	1"	LC 1	103.5	9.0	0.0
✓	SP5	A572 Gr.50	1"	LC 1	46.4	4.7	0.0
✓	SP4	A572 Gr.50	1"	LC 1	46.4	4.7	0.0
✓	GPL	A572 Gr.50	1"	LC 1	45.3	1.0	3.4
✓	Lug PL1	A572 Gr.50	1"	LC 1	45.0	0.1	3.4

Figure 9 – Evaluation of Core Plate in Corner BRB Connection - CBFEM

This variation between the two strain types as seen in Figure 8 and 9 is not only normal in detailed finite element modeling but also vital for a realistic evaluation of structural performance. The maximal equivalent strain helps detect early signs of distress or overstressing at localized zones—data that may be missed if only equivalent strain is considered. However, the equivalent strain is important in ensuring that widespread yielding does not develop in unintended locations such as gusset plates or beam flanges. When used together, they provide a robust picture of both local vulnerability and global connection behavior.

In practice, it is entirely acceptable—and even expected—that maximal and equivalent plastic strain values differ in a CBFEM analysis. Their divergence is due to mesh refinement effects, geometric irregularities, and the concentration of stress around certain features. What is critical is how this divergence is interpreted: if maximal strain is high but confined to a small area in a non-critical zone (e.g., near a weld toe in a stiffener plate), and the equivalent strain remains below permissible limits, the connection is considered structurally safe. However, if widespread high strains are observed across non-dissipative components, the design must be re-evaluated to prevent premature failure.



a.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial tension load of 300k in brace.

a.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial tension load of 300k in brace.

b.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial tension load of 400k in brace.

b.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial tension load of 400k in brace.

c.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial tension load of 500k in brace.

c.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial tension load of 500k in brace.

Figure 10 – Evaluation of Gusset Plate in Corner BRB Connection for variation of axial tension load - CBFEM

The figure 10 illustrates the CBFEM-based evaluation of a 1" A572-50 gusset plate in a Corner Buckling-Restrained Brace (BRB) connection under increasing levels of axial tension load in the brace. The six subplots (a.1–c.2) present the results in terms of von Mises stress and plastic strain contours for axial loads of 300k, 400k, and 500k.

1. a.1 and a.2 – 300k Axial Tension Load in Brace
   - a.1 (von Mises Stress): The stress distribution is mostly elastic, with peak stresses around 20–25 ksi, well below the material yield limit (50 ksi). Stress concentrations appear near the bolt holes and the gusset-beam interface, indicating expected load path zones.
   - a.2 (Plastic Strain): Minimal plastic strain is visible, mostly under 0.1%, implying that the gusset behaves elastically at this load level with a healthy reserve against yielding.

2. b.1 and b.2 – 400k Axial Tension Load in Brace
   - b.1 (von Mises Stress): Stress levels increase significantly, with localized stress reaching near 35 ksi, especially around bolt holes and connection edges, which serve as stress raisers.
   - b.2 (Plastic Strain): Plastic strain contours begin to appear near the lower bolts and edge fillets, reaching about 4–5% in some zones. This indicates incipient yielding and suggests that local plasticity is starting to occur under 400k tension.

3. c.1 and c.2 – 500k Axial Tension Load
   - c.1 (von Mises Stress): Much of the gusset is now stressed above 40 ksi, with several areas approaching or exceeding 45 ksi—the assumed material limit in CBFEM. This is a highly stressed condition, suggesting widespread yielding.
   - c.2 (Plastic Strain): Plastic strains are now fully developed in multiple regions, peaking around 11%. The plastic zones are especially prominent near bolt holes, edge corners, and gusset-beam intersections. These high values reflect extensive inelastic deformation and suggest that the gusset has reached or is close to its plastic capacity.

The gusset plate exhibits elastic behavior under a 300k axial tension load, initiates localized yielding at 400k, and develops extensive plastic deformation by 500k. The critical stress and strain concentrations are consistently observed at the bolt-to-gusset interfaces, gusset edge fillets, and gusset-to-beam transition zones. These results confirm that the gusset plate is adequately proportioned for service-level loads but requires careful evaluation under seismic design conditions, especially when higher ductility demands are anticipated.

This analysis validates the progressive yielding mechanism typical of seismic energy dissipation systems and provides engineers with insight into strain and stress localization zones. Such insights support detailing enhancements, including increased edge distances, and optimized bolt layouts, to improve overall connection resilience.

7. Verification of Resistance – Capacity of Elements Subjected to Compression Load

When a corner BRB connection is subjected to increasing axial compression loads—ranging from 300 kips to 500 kips in 10-kip increments—the structural response captured in IDEA StatiCa reveals key differences from tension loading, primarily due to geometric nonlinearity, local flexibility, and the nature of BRB design under compression.

Under compression, although the core of the BRB is designed to remain stable and resist yielding due to the surrounding casing, secondary effects become more prominent. IDEA StatiCa shows that gusset plates, especially at the corner junction between the beam and column, begin to exhibit higher local stresses and plastic strains, particularly at bolt holes, weld toes, and gusset tips. These localized zones may show early onset of yielding as compression increases, even though the brace core is still within its inelastic range.

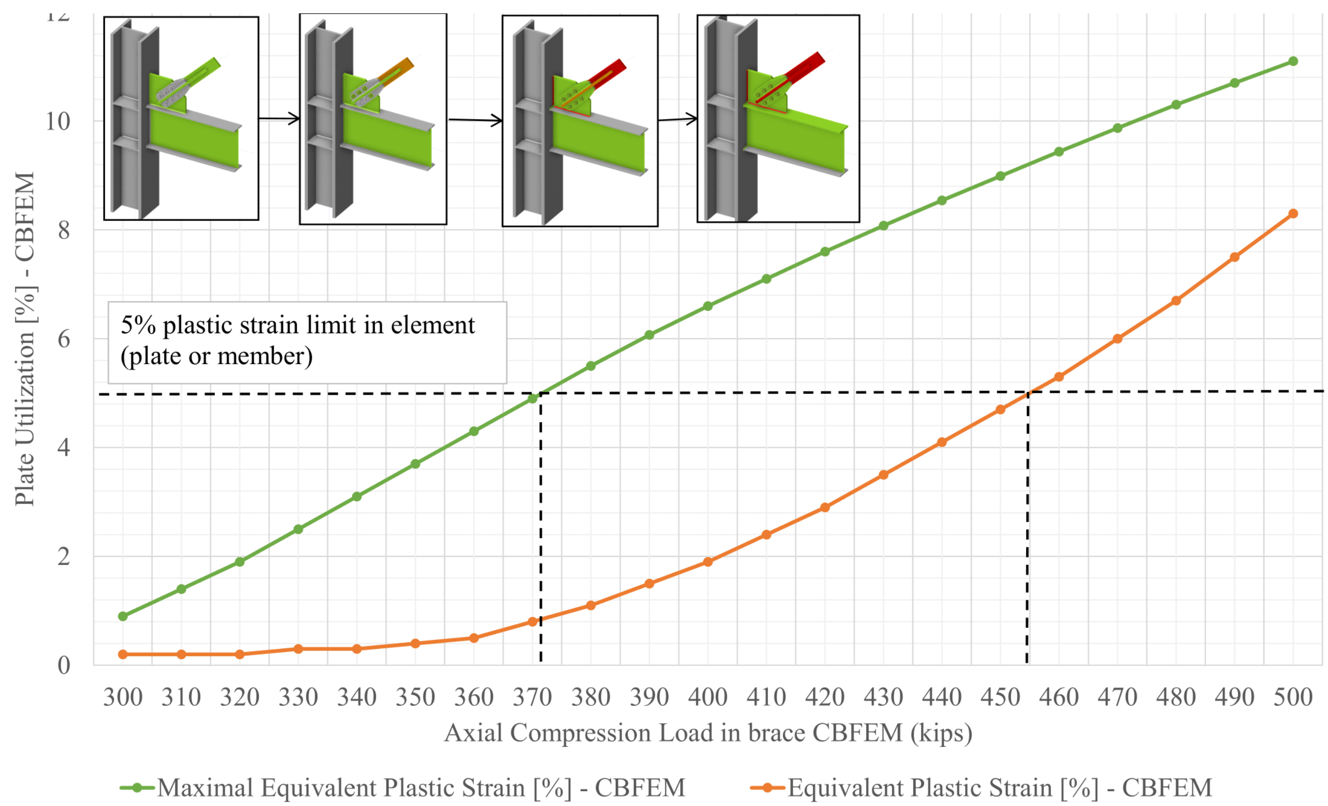
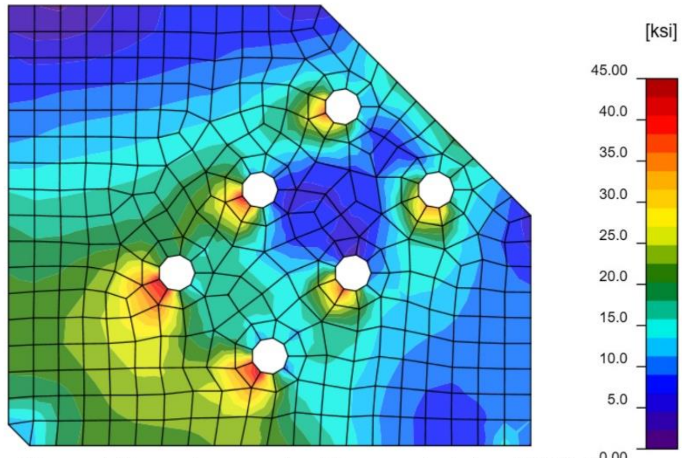


Figure 11 - Graph for Evaluation of Capacity of Elements for Compression Load in Brace– CBFEM

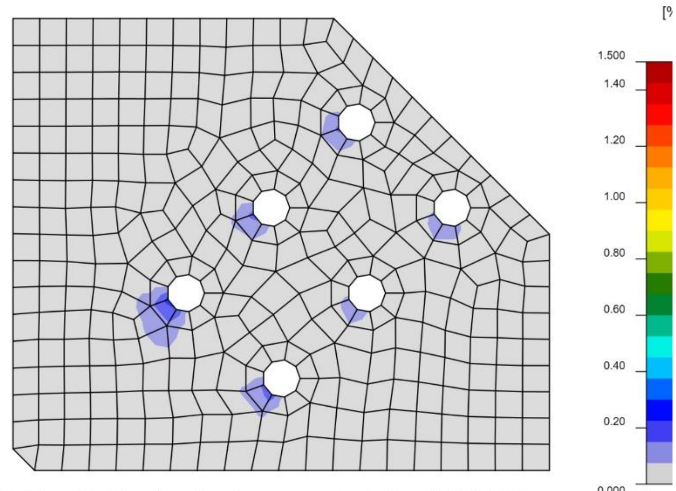
The graph as illustrated in Figure 11 presents the behavior of a single diagonal corner Buckling-Restrained Brace (BRB) connection under increasing axial compression loads ranging from 300 kips to 500 kips, evaluated using the Component-Based Finite Element Method (CBFEM) in IDEA StatiCa. Two key strain indicators are plotted: the maximal equivalent plastic strain (green curve) and the equivalent plastic strain (orange curve), both of which track the plastic deformation within the connection plates or members as the compressive force increases. The 5% plastic strain limit, widely accepted in seismic design as the upper threshold for non-dissipative elements, is marked for comparison.

The results indicate that localized yielding, represented by the maximal equivalent plastic strain, begins to exceed the 5% threshold at approximately 370 kips of axial compression. This suggests that while the BRB itself is performing within its intended ductile range, certain areas of the connection—most likely gusset corners, weld zones, or bolt hole edges—are experiencing high stress concentrations and entering plastic deformation earlier. In contrast, the equivalent plastic strain, which reflects the average distributed strain across the member or plate, increases more gradually and reaches the 5% threshold only near 455–460 kips. This divergence between the two strain values is expected, as CBFEM captures localized non-uniformities in stress distribution that traditional methods may not reflect.

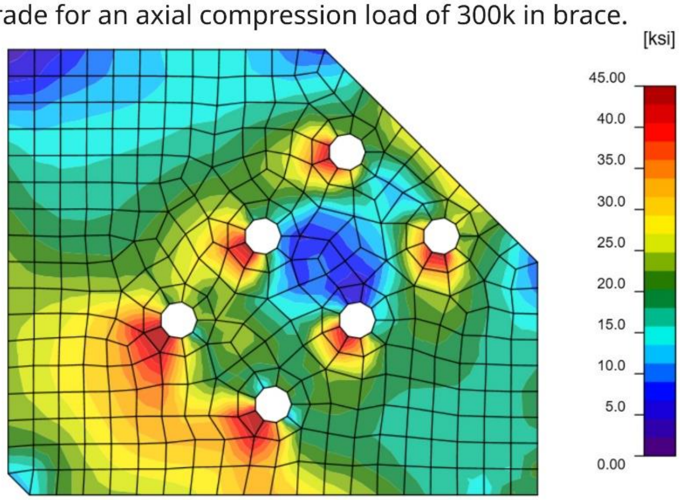
As the compression load increases, the difference between the two strain values continues to widen, highlighting the connection's sensitivity to compression-related effects such as plate flexure, geometric eccentricities, and minor out-of-plane deformations. This behavior is especially relevant in corner configurations where load paths are less direct and the connection geometry introduces additional flexibility and rotation. The progression of deformation stages is visually captured through the image series above the plot, illustrating the increasing strain and stress engagement in the gusset as the brace force intensifies.



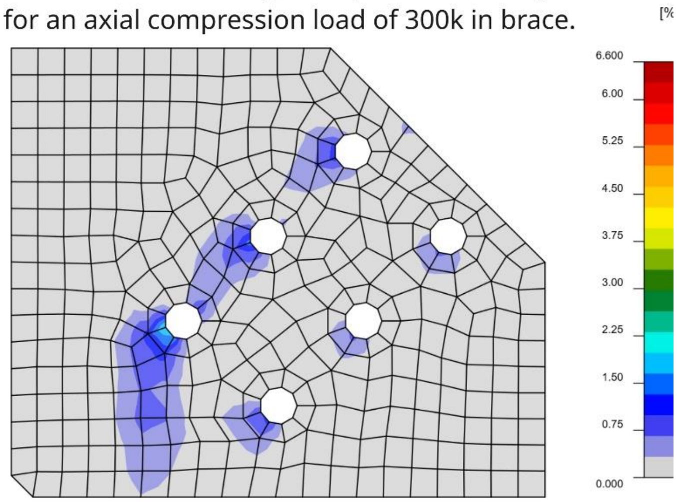
a.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial compression load of 300k in brace.



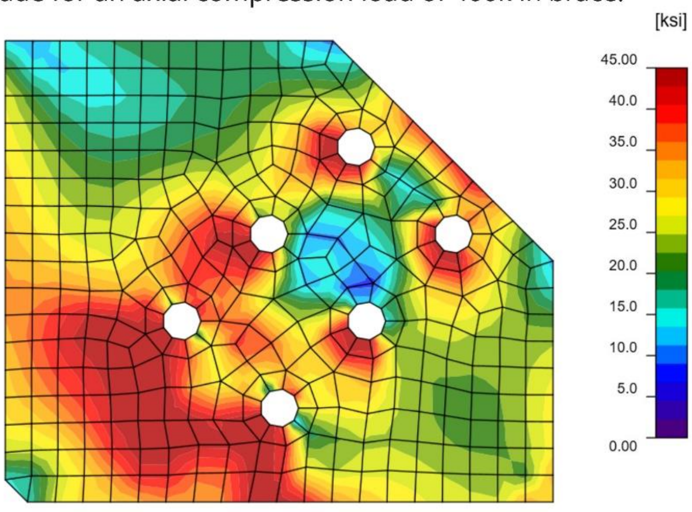
a.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial compression load of 300k in brace.



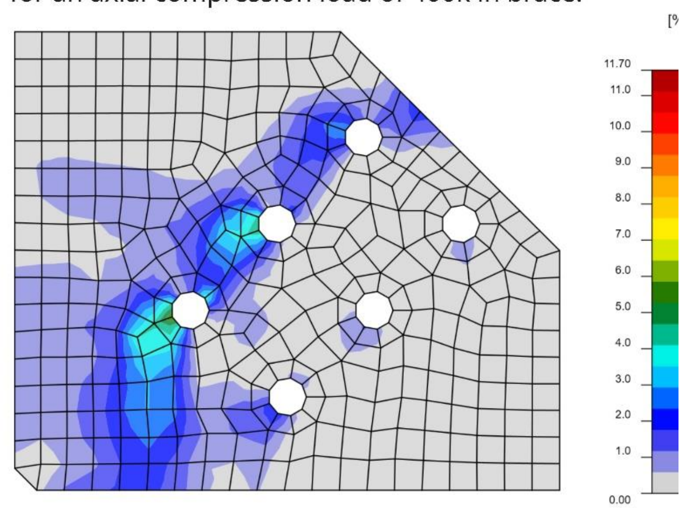
b.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial compression load of 400k in brace.



b.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial compression load of 400k in brace.



c.1] von Mises stresses in 1" gusset plate, A572-50 grade for an axial compression load of 500k in brace.



c.2] Plastic Strains in 1" gusset plate, A572-50 grade for an axial compression load of 500k in brace.

Figure 12 – Evaluation of Gusset Plate in Corner BRB Connection for Variation of Axial Compression Load - CBFEM

The figure 12 presents the results of a CBFEM-based evaluation of a 1″ thick gusset plate (A572-50 grade) in a corner Buckling-Restrained Brace (BRB) connection under varying levels of axial compression load: 300k, 400k, and 500k. The results are illustrated in terms of von Mises stress distribution (left column) and plastic strain contours (right column), offering insight into the gusset's performance as loading intensifies.

1. a.1 and a.2: 300k Axial Compression Load in Brace
   - Stress Behavior: At 300k, the von Mises stress remains within the elastic limit (< 36 ksi), distributed relatively uniformly across the gusset plate.
   - Strain Behavior: Plastic strain is negligible (< 1.5%), indicating fully elastic behavior with no localized yielding.
   - The gusset plate is structurally safe and well within elastic range for this load level.

2. b.1 and b.2: 400k Axial Compression Load in Brace
   - Stress Behavior: At 400k, elevated stress concentrations begin to appear near the bolt holes and gusset-beam interface, approaching the yield stress of A572-50 steel (50 ksi).
   - Strain Behavior: Localized plastic strains up to 6% emerge, particularly around bolt interfaces and plate corners.
   - The gusset begins to yield in localized zones, especially at stress transfer points. These early yielding regions suggest the need to monitor ductility demands for seismic performance.

3. c.1 and c.2: 500k Axial Compression Load in Brace
   - Stress Behavior: Significant portions of the gusset experience von Mises stress between 35–45 ksi, with some areas nearing the material yield limit.
   - Strain Behavior: Extensive plastic deformation is visible, with plastic strain exceeding 10% in some areas, especially near bolt holes, plate corners, and the gusset-to-beam transition zone.
   - The gusset exhibits large-scale inelastic behavior. While this is acceptable for energy dissipation under seismic events, it may prompt design enhancements such as increased edge distance, thicker gusset plates, or optimized bolt configurations for enhanced performance.

Critical stress and strain concentrations in the gusset plate are consistently observed at three primary locations: the bolt-to-gusset interfaces, the gusset corners near edge fillets, and the transition zone between the gusset and adjoining framing members. The CBFEM analysis demonstrates that the gusset plate behaves elastically under service-level loading conditions (300 kips) and begins to exhibit progressive yielding as the axial load increases to 400–500 kips, which aligns with the expected energy dissipation mechanism of Buckling-Restrained Braced Frames (BRBFs). These findings confirm the gusset plate's adequacy for inelastic seismic demand while also highlighting regions susceptible to local plasticity.

The behavior of a corner Buckling-Restrained Brace (BRB) connection under axial compression differs significantly from its response under axial tension. Compression loading results in a broader and less uniform stress distribution, a higher risk of out-of-plane deformation in the gusset plate, and a faster onset of localized plastic strains. In contrast, tension loading leads to more predictable axial load paths, characterized by uniform engagement of bolts and welds, which makes the response easier to interpret and control.

Under axial tension, the gusset plate exhibits a relatively uniform von Mises stress distribution, with stress concentrations localized around bolt holes and plate edges. This reflects a predictable in-plane yielding behavior, where plastic strains develop gradually with increasing load. Even at 500 kips, these strains remain controlled and localized, indicating a ductile response—a behavior aligned with BRB performance objectives under seismic loading, where energy dissipation through yielding is preferable to brittle failure.

In contrast, axial compression introduces complex, asymmetric stress states. Stress tends to concentrate more sharply near the gusset-to-beam transition zone and around the bolt group, where the compressive force is applied. At higher load levels (e.g., 500 kips), the plastic strain field becomes more widespread compared to the tension case. This is attributed to localized bearing effects, plate flexure, and potential out-of-plane deformation modes such as gusset buckling or crippling.

The failure mechanisms for the two load cases – Tension load in brace and Compression load in brace also differ significantly. In tension load case, critical limit states include net section rupture, bolt tear-out, and localized yielding around bolt lines. In compression load case, concerns shift toward buckling, local crushing, instability at gusset edge fillets, and crippling of the gusset plate or adjacent web elements. As a result, different design strategies are required to adopted:

- For tension load case, the focus should be on maintaining adequate edge distances, bolt spacing, and smooth fillet transitions to reduce stress concentrations.
- For compression, the emphasis should be on ensuring gusset compactness, increasing bearing length, and incorporating stiffeners to manage local instability and distribute compressive forces effectively.

Importantly, in Buckling-Restrained Braced Frames (BRBFs), the core of the BRB is designed to yield in both tension and compression while preventing global buckling. This fundamental feature is what enables BRBFs to provide superior seismic performance and energy dissipation. Although the CBFEM analysis in IDEA StatiCa confirms brace load transfer and associated yielding behavior in the gusset plate, it does not simulate the full BRB element, and thus cannot capture the following:

- The yielding length of the BRB core,
- The confinement effects of the casing,
- The strain-hardening response of the steel core.

Therefore, while the CBFEM results support the expected compression yielding behavior of the BRB, the evaluation is limited to local effects at the gusset and joint interface. To fully validate brace core yielding for axial compression load, we must rely on:

- Manufacturer test data results performed in full scale labs (e.g., from CoreBrace),
- Nonlinear modeling in software such as OpenSees or ABAQUS, and
- Global structural analysis tools like SAP2000, ETABS, or Perform3D.

In conclusion, for the considered example the gusset plate demonstrates a stable and ductile response under tension, while compression loading introduces additional challenges, such as higher plastic strain concentrations, stress asymmetry, and local instability risks.

8. Verification of Resistance - Capacity of Connections

For the given bracing connection, multiple structural components—including gusset plates, beams, stiffeners, and brace-end elements—are interconnected using fillet welds of various sizes and lengths and also bolted connection. These welds play a crucial role in transferring axial and shear forces across the connection interfaces and ensuring structural integrity under seismic loading. After assessing the strength and deformation capacities of the primary elements such as plates and members, a detailed evaluation of the connections – bolted and welded was performed for axial tension load in brace member using the same Component-Based Finite Element Method (CBFEM) model within IDEA StatiCa. This approach allowed for consistency in material properties, geometry, and loading conditions.

In the weld analysis, particular attention was given to regions where welds exhibited high stress concentrations. The weld utilization ratio—an output parameter in IDEA StatiCa—was used to identify critical zones. Locations where the weld utilization reached or exceeded 100% were flagged for further evaluation, as these welds are fully mobilized and potentially critical to the performance of the connection.

The evaluation of bolted connections in a corner bracing configuration is critical to ensuring seismic performance. In the CBFEM analysis using IDEA StatiCa, bolted joints—typically between gusset plates and lug plates or BRB end plates—are assessed for their ability to transfer axial forces during both tension and compression loading cycles.

Several factors are considered in the bolt evaluation, including bolt layout, edge distances, spacing, and hole types (standard, oversized, or slotted). Preload conditions, such as slip-critical or bearing-type, also influence performance. The software checks multiple failure modes automatically in accordance with AISC provisions. These include bolt shear, plate bearing, block shear, and combined tension-shear interaction.

In seismic design, bolts are expected to remain elastic even under overstrength-level brace forces. Therefore, bolt utilization must be ideally limited to 90% to account for uncertainties and ensure safety. If utilization exceeds 100%, it indicates a potential failure mechanism. In such cases, the design can be adjusted by increasing bolt diameter, changing the bolt grade (e.g., from A325 to A490), revising the bolt pattern, or increasing gusset plate thickness to reduce bearing stresses.

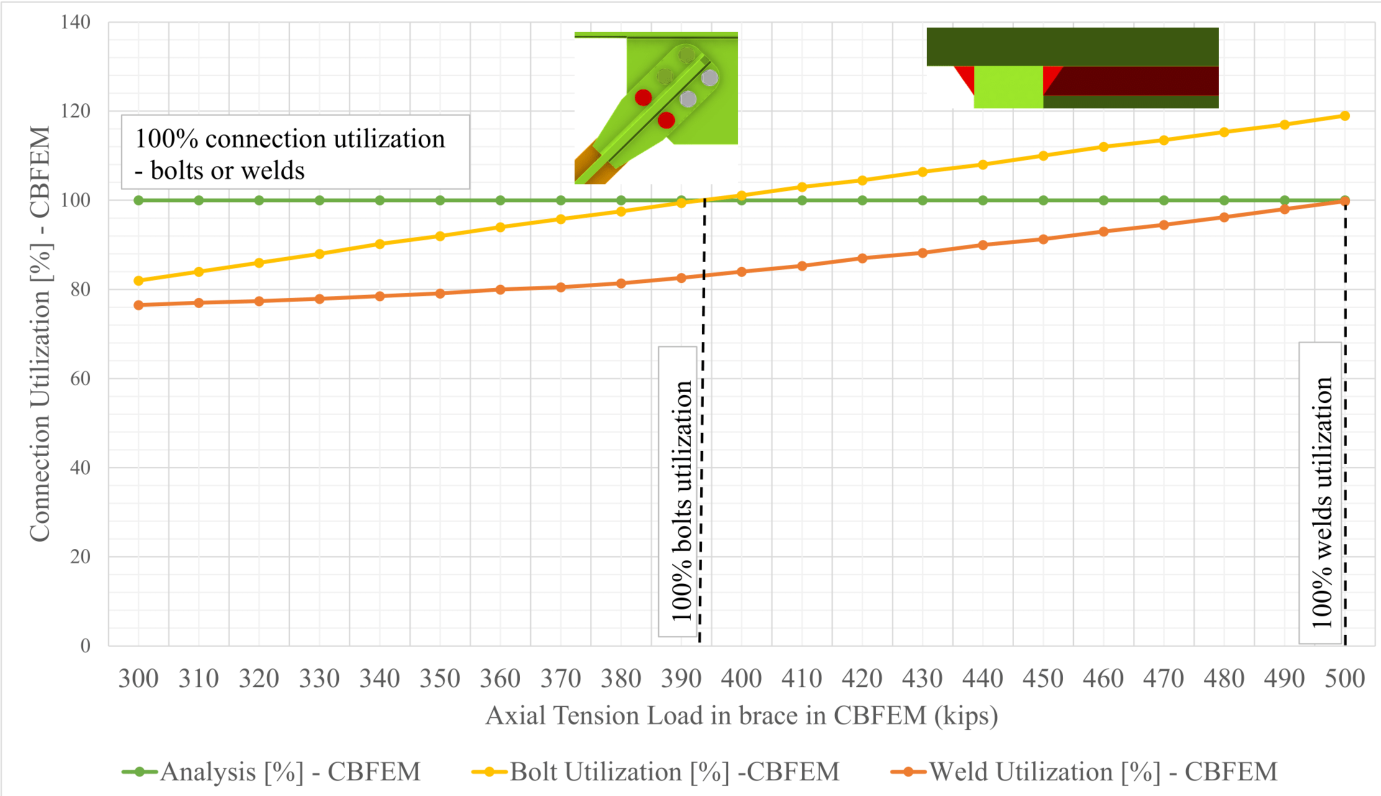


Figure 13 - Graph for evaluation of capacity of connection – CBFEM

Figure 13 illustrates the connection utilization behavior of a corner BRB connection under increasing axial tension loads in the brace, as analyzed using the Component-Based Finite Element Method (CBFEM) in IDEA StatiCa. The graph plots the utilization of key connection components—namely bolts and welds—as a function of the applied axial tension load, ranging from 300 kips to 500 kips. The vertical axis shows connection utilization in percentage, with 100% representing the limit of allowable capacity for each element. Three curves are plotted: overall analysis utilization (green), bolt utilization (yellow), and weld utilization (orange), each reflecting the performance of a different part of the connection.

From the plotted results, it is evident that bolt utilization increases steadily with the axial load and reaches 100% capacity at approximately 390 kips. This indicates that the bolts become the first critical component in the load path and could potentially govern the design if the connection is expected to resist brace forces beyond this value. Weld utilization, on the other hand, starts at a lower initial percentage but increases more gradually, reaching 100% only near 495–500 kips. This delayed saturation suggests that welds are not the governing failure mode until the brace is subjected to near-maximum axial loads, allowing some margin for redistribution of stresses within the connection up to that point.

The green line representing the overall connection utilization remains near 100% throughout the entire range, as CBFEM assumes full force transfer through the joint and evaluates the most critical stress path. However, this does not necessarily mean the entire connection is failing; rather, it implies that one component (in this case, bolts or welds) is being utilized to its full allowable capacity based on design code checks. This highlights the importance of evaluating each component individually, especially in seismic design, where capacity protection of non-dissipative elements is essential.

The graph also includes informative inset 3D views showing the brace-to-gusset bolted connection and a typical weld interface. These visualizations help contextualize the utilization data and provide clarity on where bolt or weld failures may initiate.

9. Code check of connection in CBFEM

Proportioning braces based on their adjusted strength is a fundamental requirement in the seismic design of Buckling-Restrained Braced Frames (BRBFs), as recommended by AISC 341-16. According to this provision, the adjusted tensile strength of the brace is defined as:

$$P_{tension} = \omega R_y P_{ysc}$$

and the adjusted compressive strength is given by:

$$P_{compression} = \beta \omega R_y P_{ysc}$$

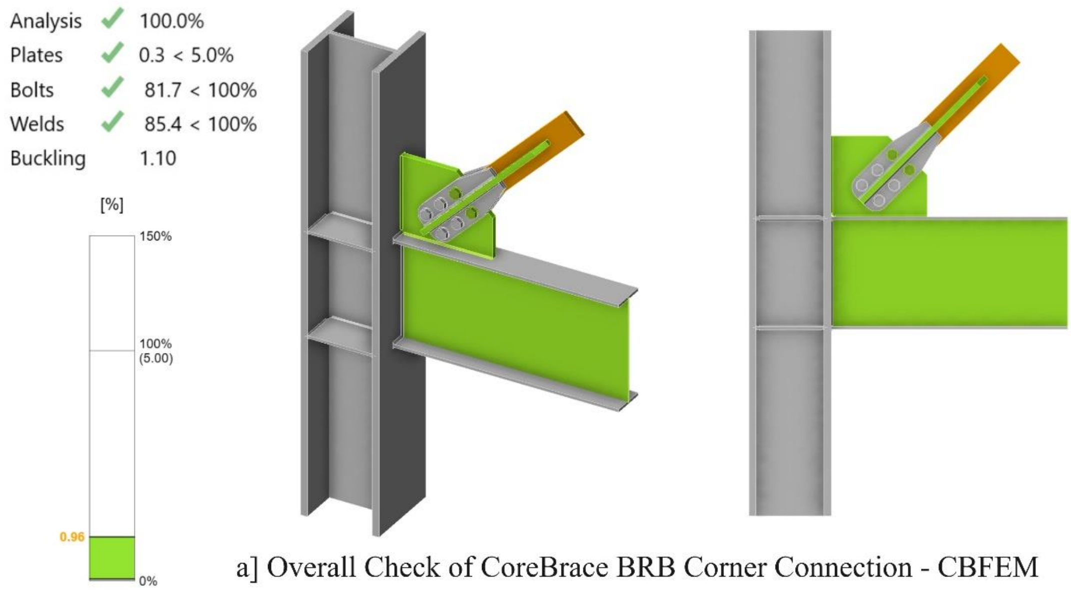
The required adjustment factors are typically provided by the BRB manufacturer (e.g., CoreBrace), and each parameter is defined as follows:

- β is compression strength adjustment factor
- ω is the strain hardening adjustment factor
- R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y
- P_{ysc} is the axial yield strength of steel core, ksi (MPa), calculated as $P_{ysc} = F_{ysc}.A_{sc}$
- F_{ysc} is the yield strength of steel core in ksi (typically 38 to 46 ksi, per industry standards)
- A_{sc} is the area of steel core in in^2

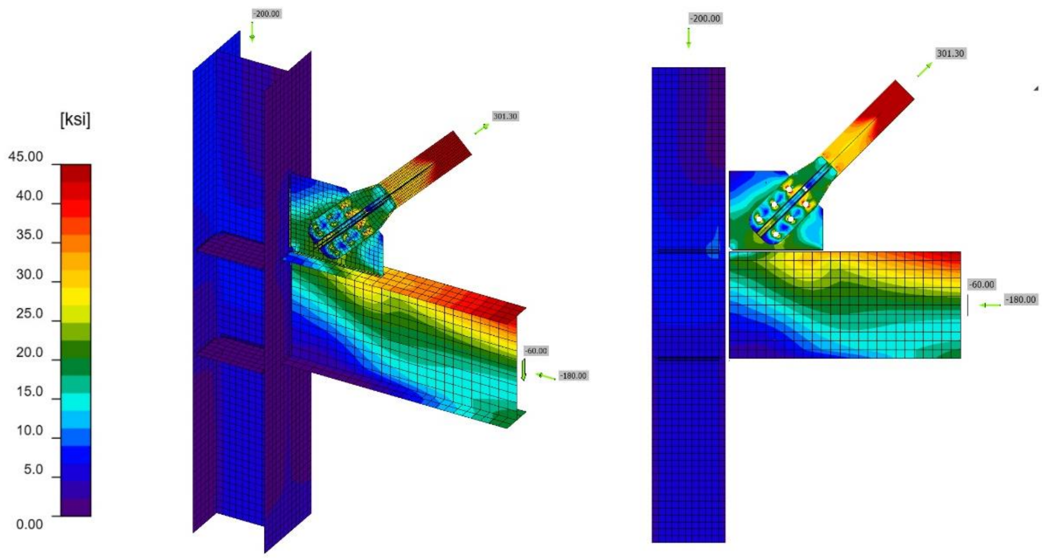
As specified in Section F4.3 of AISC 341-16, the required strength of the surrounding frame elements—such as columns, beams, struts, and their associated connections—must be determined using capacity-based seismic forces. These forces are calculated assuming that each brace achieves its adjusted strength in either tension or compression, depending on the governing load condition, and they explicitly exclude gravity loads from the assessment.

In alignment with this provision, the current study evaluates a CoreBrace BRB corner connection using the Component-Based Finite Element Method (CBFEM) implemented in IDEA StatiCa, in accordance with AISC 360-16 and AISC 341-16 requirements. The Buckling-Restrained Brace (BRB) is modeled as a dissipative component, concentrating inelastic deformation within the yielding steel core. All other elements—gusset plates, bolts, welds, beams, and columns—are treated as non-dissipative, remaining elastic throughout the seismic event to ensure reliable force transfer and structural integrity.

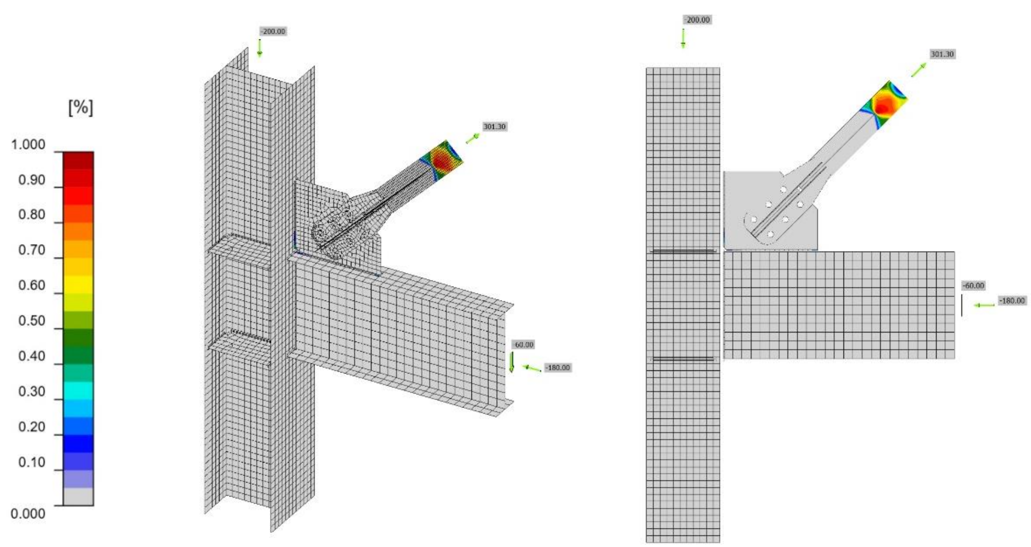
The connection was subjected to axial brace forces corresponding to the overstrength-level design loads, which incorporate the required amplification factors: β (compression strength adjustment), ω (strain hardening factor), and R_y (material overstrength factor). The application of these amplified loads and the resulting connection behavior are illustrated in Figure 14, providing insight into the effectiveness of the connection under seismic demand as per capacity design principles.



a] Overall Check of CoreBrace BRB Corner Connection - CBFEM



b] von Mises stresses of CoreBrace BRB Corner Connection - CBFEM



c] Plastic strains of CoreBrace BRB Corner Connection - CBFEM

Figure 14 – Evaluation of Corner CoreBrace BRB Connection for Design Loads Using CBFEM

The results from Figure 14 are organized into three distinct parts, each assessing a key performance aspect of the CoreBrace BRB corner connection—namely, code compliance, stress distribution, and plastic strain behavior:

- Part (a): Overall Code Compliance

  The CBFEM analysis confirms that the connection meets all limit state criteria, with the model converging at 100% completion. Plate plastic strains ranged between 0.3% and 5.0%, remaining within the permissible limit for non-dissipative elements in seismic design. Bolt utilization was recorded at 81.7%, and weld utilization at 85.4%, both comfortably below the 100% threshold, indicating sufficient safety margins in connection detailing.

- Part (b): Stress Distribution-von Mises

  The von Mises stress contours display well-distributed stress flow through the gusset-to-brace interface, bolt regions, and beam-column connection zones. Peak stress values remain under 45 ksi, which aligns with the yield strength of ASTM A572 Grade 50 steel. The absence of abrupt stress concentrations and the smooth transition of force paths suggest efficient load transfer and effective geometric and material detailing of the connection.

- Part (c): Plastic Strain Localization

  The plastic strain contours reveal that inelastic deformation is confined to the BRB core end, as intended by capacity design philosophy. All surrounding elements—including the gusset plate, beam flange, column face, and weld groups—remain within the elastic range, confirming that they act as reliable, non-dissipative force-transferring components. No significant plasticity was observed in bolts or welds, underscoring the adequacy of their sizing and layout.

The CBFEM-based evaluation demonstrates that the CoreBrace BRB corner connection is structurally sound, code-compliant, and suitably detailed for seismic applications. The results affirm that the design successfully isolates yielding to the BRB core while maintaining elastic performance in all other components, thereby adhering to the principles of capacity design and seismic resilience.

10. Evaluation of Limit State for Tension Load in Brace - CBFEM

The evaluation of the Buckling-Restrained Brace (BRB) corner connection in this study was conducted using two complementary approaches: traditional code-based analysis per AISC 360-16 (LRFD) and advanced numerical modeling via the Component-Based Finite Element Method (CBFEM) in IDEA StatiCa Version 25.0. The objective was to verify the seismic performance of the connection under capacity-level axial loading by assessing critical limit states, including tensile yielding, bolt and weld strength, and localized plasticity in gusset plates. This dual-method approach enables a comprehensive assessment by comparing theoretical strength predictions against detailed stress-strain responses derived from finite element simulations.

In the AISC-based analytical method, connection components were evaluated using Load and Resistance Factor Design (LRFD) provisions, with each element checked against codified strength limits to ensure safety and compliance. Concurrently, the CBFEM simulation modeled the connection under incrementally increasing axial brace loads, capturing the development of von Mises stress and equivalent plastic strain in gusset plates, bolts, and welds. The maximum load capacity was identified through a stepwise loading procedure (in 10 kip increments) until IDEA StatiCa signaled a transition from a safe to an unsafe condition—typically when plastic strain in non-dissipative components exceeded the allowable 5% limit. This iterative verification process provided a direct, visual, and quantitative measure of the connection's strength, offering a robust validation of the analytical results derived from AISC provisions.

10.a. Tensile Yielding in Gusset Plate

One of the critical limit states assessed in this study was tensile yielding of the gusset plate, particularly in lug-type BRB connections. This limit state considers yielding across the gross cross-sectional area of the gusset plate perpendicular to the applied axial force from the brace. Unlike tensile fracture checks, where bolt holes are deducted from the net area, tensile yielding assumes a uniform stress distribution and initiates across the entire gross width of the plate. As per AISC 360-16, Section D2(a), the design strength for yielding in tension is expressed as:

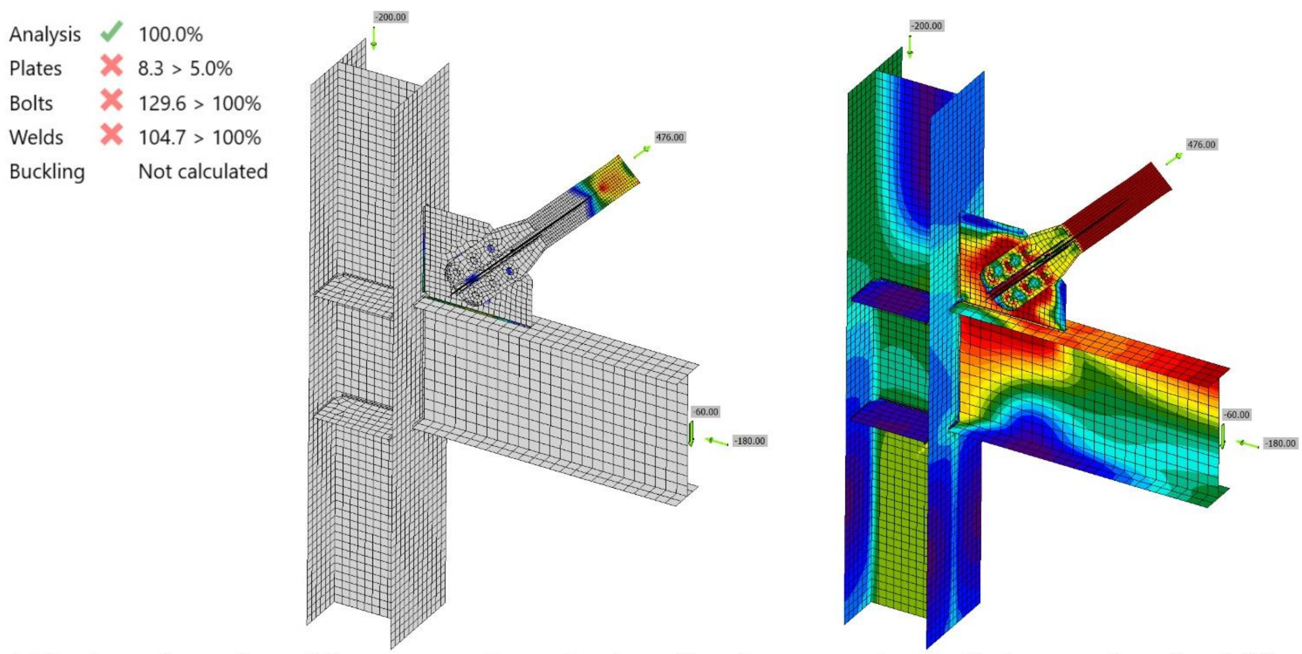
$$\phi R_n = \phi_t F_y A_g$$

Where:

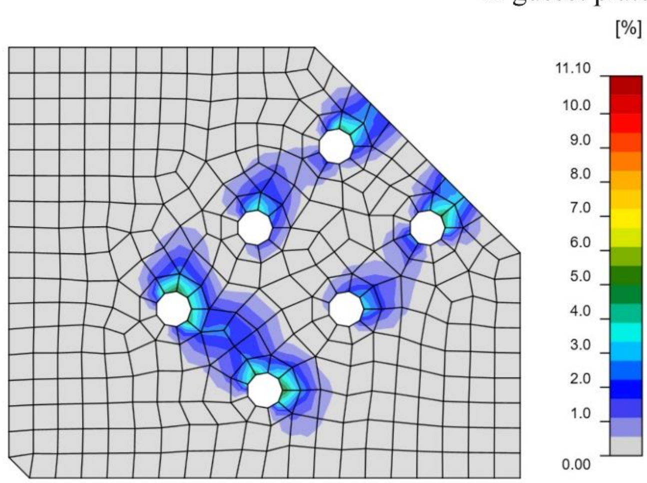
- ϕ_t =0.90: Resistance factor for yielding (LRFD)
- F_y is the specified minimum yield strength of gusset plate material
- A_g *is the* gross area of the tension plane, calculated as: $A_g = tw$
- t is the thickness of gusset plate
- w is width across the gross yielding plane

This equation provides a conservative estimate of the gusset's capacity, ensuring that yielding, if it occurs, is well within ductile limits and does not compromise the integrity of the surrounding connection elements.

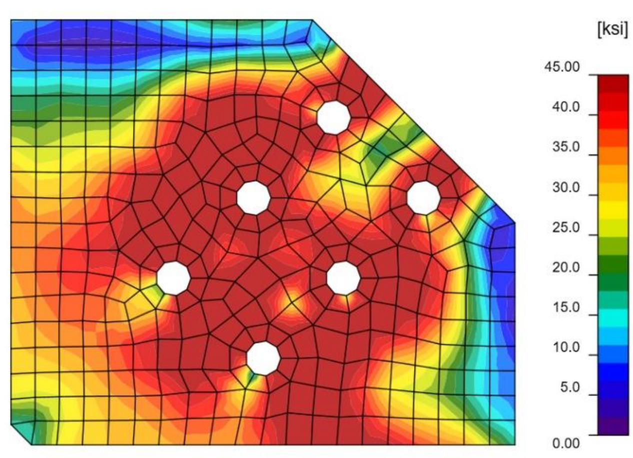
The limit state of tensile yielding in the gusset plates was identified in the CBFEM analysis at an axial tension load of 476 kips in the bracing member, as illustrated in Figure 14. This result exceeds the calculated capacity of 450 kips based on AISC provisions, indicating a deviation of approximately 6%. The slight increase in capacity observed in the CBFEM model can be attributed to its ability to capture localized stress redistribution and strain hardening effects, which are not considered in the uniform stress-based assumptions of traditional AISC calculations. While tensile yielding is generally not the governing limit state in BRBF corner connections — where buckling, bolt shear, or weld rupture often dominate—it remains a critical check to ensure code compliance and validate the integrity of the gusset under overstrength-level brace forces.



a) Plastic strains and von Mises stresses for evaluation of bracing connection for limit state of tensile yielding in gusset plate - CBFEM



b) 5% Plastic Strains in Gusset Plate -CBFEM



c) von Mises stresses in Gusset Plate -CBFEM

Figure 15 – Evaluation of Limit State of Tensile Yielding in Gusset Plate – CBFEM

Figure 15 illustrates a Component-Based Finite Element Method (CBFEM) assessment of a corner BRB connection, specifically evaluating the gusset plate for the limit state of tensile yielding under axial tension loading of 476 kips. This evaluation is essential to ensure that the gusset plate remains within acceptable performance limits and does not undergo excessive plastic deformation in seismic or high-force scenarios.

- Figure a presents two visualizations of the BRB corner connection under tension, illustrating how stress propagates through the gusset plate and into the surrounding beam and column members. The tension force introduced by the BRB is effectively transmitted via the bolted lug plate into the gusset plate. The stress flow is shown to intensify near the brace-to-gusset interface, with a smooth and continuous distribution extending toward the beam flange and column face. This aligned stress trajectory confirms efficient and rational force transfer, with no signs of stress discontinuity or abrupt concentration, indicating that the connection is structurally well-integrated and consistent with the principles of good seismic detailing.
- Figure b illustrates the plastic strain distribution in the gusset plate, with observed strain levels reaching approximately 5%, which aligns with the seismic design threshold for non-dissipative components. The localized plastic strains are concentrated along the tension plane perpendicular to the brace axial force, indicating expected yielding behavior. These strain zones correspond well with the gross section of the gusset plate as defined in tensile yielding checks under AISC 360-16, Section D2. Importantly, the plastic deformation remains confined to the gusset plate, without propagating into the adjacent beam or column members, confirming that inelastic action is successfully restricted to the intended connection component as per capacity design philosophy.
- Figure c presents the von Mises stress contours within the gusset plate, highlighting peak stress regions ranging from 35 to 45 ksi. These elevated stress zones are primarily concentrated along the central axis of the plate, aligned with the applied axial tension from the BRB, and are particularly pronounced around the bolt hole regions. The stress magnitudes closely approach the yield strength of ASTM A572 Grade 50 steel ($F_y \approx 50$ ksi), indicating that the gusset plate is operating near its elastic limit. Notably, the stress distribution is uniform across the gross section, with no abrupt spikes or edge concentrations, suggesting stable yielding behavior without signs of local instability or tearing. This confirms that the gusset plate is effectively engaged in axial load transfer in accordance with expected performance criteria.

The AISC methodology for evaluating tensile yielding in gusset plates is fundamentally based on the assumption of uniform stress distribution across the gross section of the plate. This assumption facilitates straightforward hand calculations and yields a conservative design by defining the yield limit state at the onset of first yield. While effective for standardized scenarios, this method does not account for stress redistribution or local stress concentrations that may develop in thick, stiffened, or irregularly shaped gusset plates.

In contrast, the Component-Based Finite Element Method (CBFEM) implemented in IDEA StatiCa captures the true behavior of the connection by discretizing it into a refined mesh of shell elements. This allows the software to simulate stress gradients, localized plasticity, and redistribution of forces, resulting in a more realistic assessment of the gusset plate's response under tension. Such detailed modeling often reveals a slightly higher usable capacity compared to the conservative estimates of the AISC approach, especially in cases where stiffness and geometry enhance load sharing.

A critical distinction lies in how yielding is defined. AISC designates yielding at the point where the stress first reaches the material's yield strength (F_y), regardless of the strain magnitude. On the other hand, CBFEM permits limited plastic deformation—typically up to 5% plastic strain—before flagging a failure condition. This aligns with performance-based design philosophies and remains within acceptable strain thresholds for non-dissipative elements, particularly in seismic applications, where controlled ductility is desirable.

In summary, while AISC provisions offer a conservative and codified baseline, the CBFEM method enables a performance-based, detailed verification that better reflects actual load paths and stress behaviors in complex connection geometries. Together, these methods provide complementary tools for safe and reliable gusset plate design—balancing simplicity with fidelity.

10.b. Block Shear Rupture in Gusset Plate

Block shear rupture is a critical limit state for gusset plates connected to Buckling-Restrained Braced Frame (BRBF) members via bolted lug plates. This failure mode arises due to the combined action of axial tension and shear forces that develop around the bolt group under seismic or axial loading. A portion of the gusset plate may tear out along a defined failure surface that typically includes a shear path parallel to the bolt lines and a tensile path perpendicular to the direction of the applied load.

As depicted in Figure 16, the block shear failure path reflects the interaction between tensile rupture and shear yielding or rupture, necessitating a dual check of both mechanisms per AISC 360-16 provisions. Accurate evaluation of this limit state is vital to ensure the integrity, ductility, and seismic reliability of BRBF corner connections, particularly where lug-type detailing concentrates load effects around the bolt pattern.

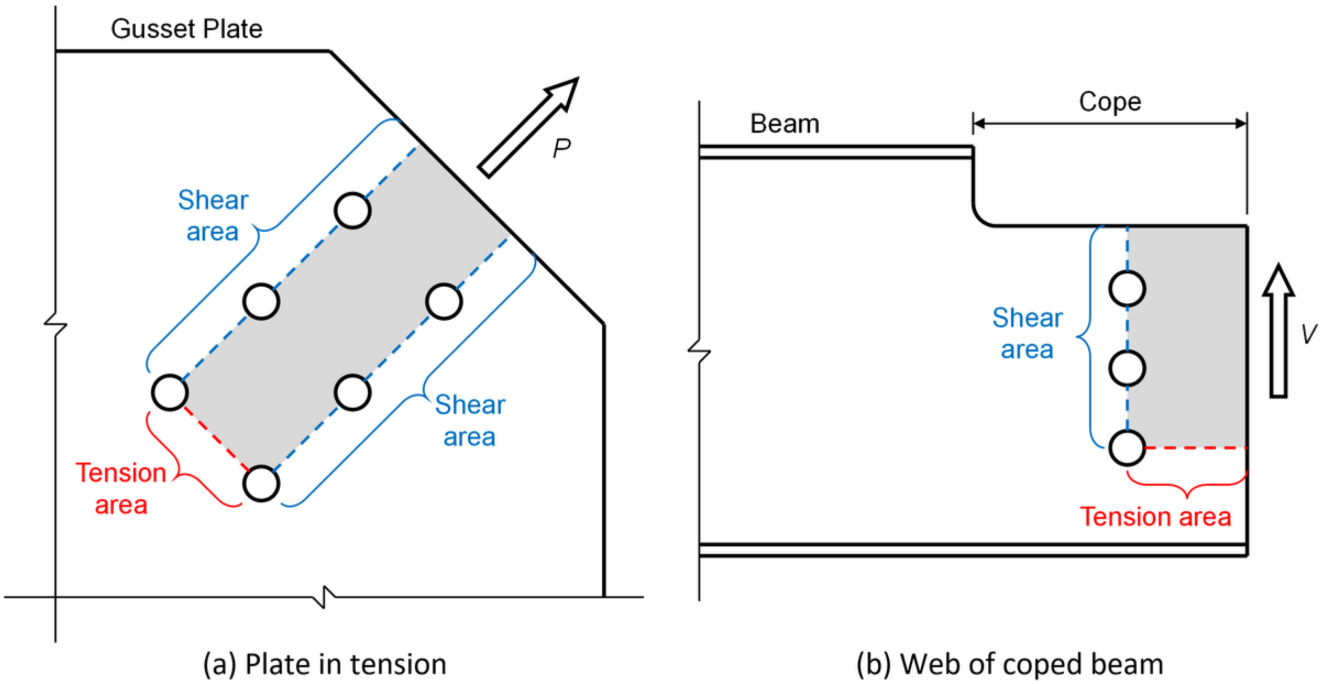


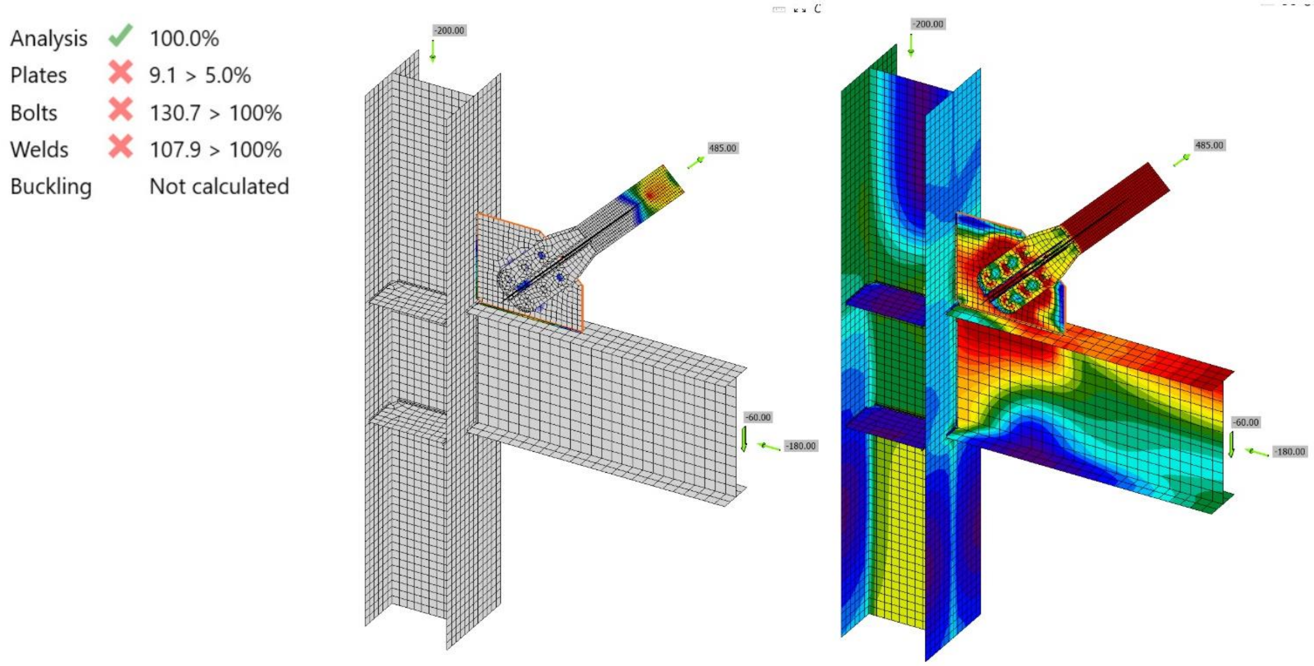
Figure 16 – Block Shear Rupture Pattern in Bolted Connections (IDEA StatiCa, n.d.)

To evaluate the block shear rupture limit state for the corner BRB connection, both AISC 360-16 provisions and CBFEM analysis were utilized. According to AISC 360-16 Section J4.3, the block shear strength is governed by:

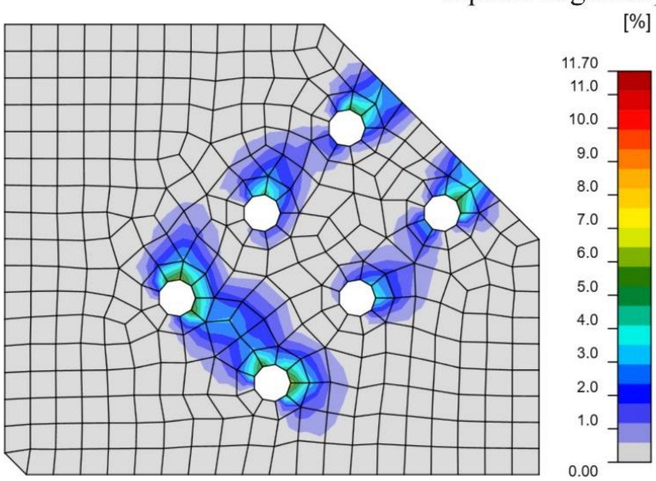
$$\Phi.R_n=\phi min[0.6F_uA_{nv}+F_uA_{nt},\ 0.6F_yA_{gv}+F_uA_{nt}]$$

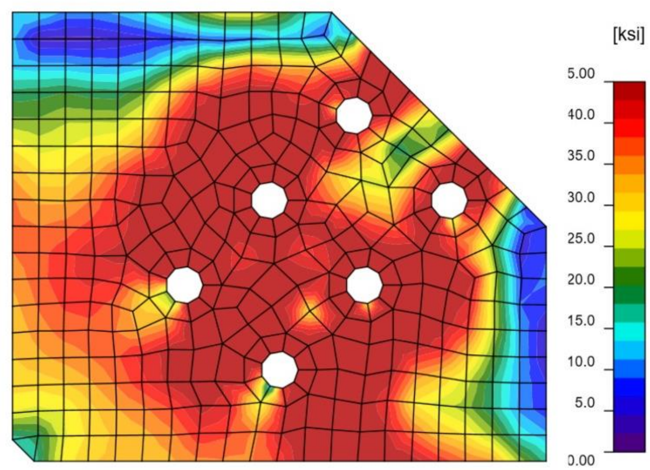
Where:

- A_{nv} is Net area in shear
- A_{nt} is Net area in tension
- A_{gv} is Gross area in shear
- F_y, F_u are Yield and ultimate strengths of the gusset plate material
- ϕ=0.75 is the resistance factor for rupture



a) Plastic strains and von Mises stresses for evaluation of bracing connection for limit state of block shear rupture in gusset plate - CBFEM





b) 6.1% Plastic Strains in Gusset Plate -CBFEM c) von Mises stresses in Gusset Plate -CBFEM

Figure 17 – Evaluation of limit state of block shear in gusset plate – CBFEM

Figure 17 illustrates a detailed Component-Based Finite Element Method (CBFEM) evaluation of a BRB corner bracing connection, conducted to assess the onset and progression of block shear rupture in the gusset plate. The connection is subjected to a critical axial load of 485 kips, which corresponds to the capacity at which localized rupture mechanisms are observed.

- Figure (a) presents both 3D and 2D CBFEM visualizations showing stress propagation through the gusset and beam interface. The stress contours clearly define a potential block shear failure path—shear planes adjacent to bolt lines and a tension path across the net section toward the gusset plate edge. This matches the classical block shear rupture mechanism observed in practice.
- Figure (b) highlights the plastic strain concentrations in the gusset plate. Strains exceeding 5% appear along bolt lines and perpendicular to the brace force direction, confirming combined shear and tensile yielding.

These strain zones coincide with the theoretical rupture surface, reinforcing the accuracy of the CBFEM model in predicting failure behavior.

- Figure (c) illustrates the von Mises stress distribution under the same axial load. Stress magnitudes in excess of 40 ksi are concentrated near bolt holes and extend toward the plate edge. These high-stress regions indicate critical zones where rupture is imminent under further loading.

In the CBFEM study, the gusset plates were analyzed under incrementally increasing axial loads applied through the brace members. The following key findings were made, as shown in Figure 16:

- Block shear rupture initiated at an axial load of approximately 485 kips, which is about 1% higher than the 480 kips predicted by AISC provisions, indicating close agreement but slightly greater capacity in the finite element model.
- The observed rupture path involved shear failure along the bolt lines and tensile failure across the net section from the bolt holes to the gusset edge.
- Localized plastic strain results corroborated the formation of this rupture path, demonstrating the combined action of shear and tension, which defines block shear failure.

This limit state evaluation demonstrates the superior predictive ability of CBFEM in modeling non-uniform stress distributions and interactive failure mechanisms. Unlike traditional AISC design equations—which assume independent, elastic-based shear and tension planes—CBFEM incorporates plastic deformation (up to ~5%), geometric irregularities, and actual material stress-strain behavior. This enables a more nuanced, performance-based assessment of connection capacity.

Moreover, the finite element model captures realistic effects of plate thickness, bolt spacing, and edge distances, resulting in a more accurate and potentially more efficient design than conservative hand calculations alone. While block shear may not always govern the overall design—especially when yielding or gross section rupture prevails—it remains a critical check, particularly in bolted gusset-to-lug plate connections, where detailing plays a decisive role in seismic performance.

10.c. Bolt Shear

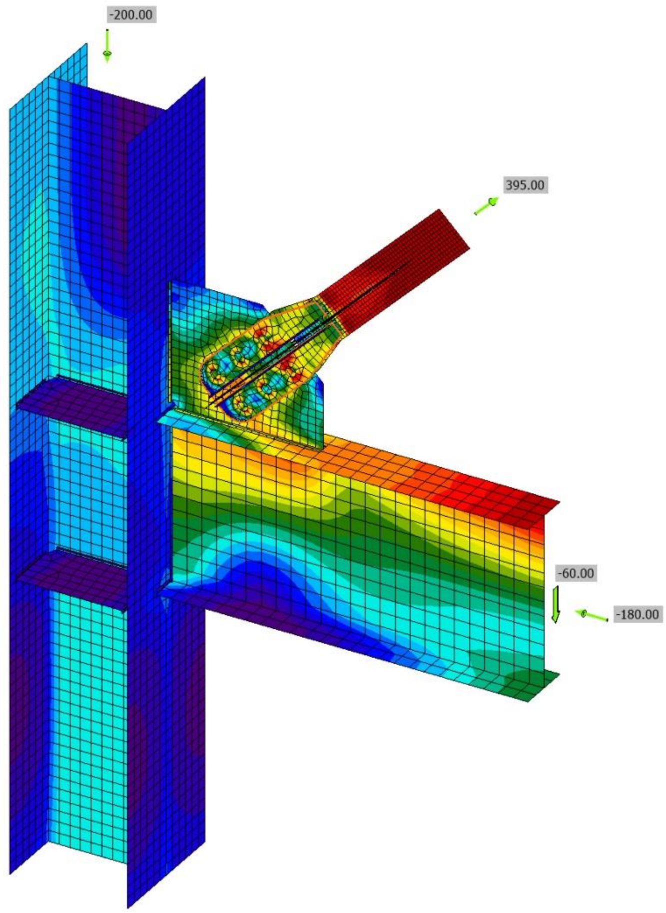
The bolt shear limit state addresses the potential for failure due to excessive shear stress acting across a bolt's cross-sectional area. This mode of failure is particularly significant in Buckling-Restrained Braced Frame (BRBF) systems, where bolts are used in high-demand regions such as gusset-to-brace, lug-to-gusset, and gusset-to-beam connections. In this study, the focus is placed on evaluating the shear capacity of bolts connecting the lug plate to the gusset plate.

Per seismic design requirements in AISC 341-16, bolts used in Seismic Force-Resisting Systems (SFRS) must be pretensioned and designed as slip-critical connections to prevent unintended slip under cyclic seismic loading. Furthermore, when bolts are subjected to combined tension and shear, their interaction must satisfy the criteria outlined in Equation J3-6 of AISC 360-16, ensuring safe performance under multi-axial load combinations.

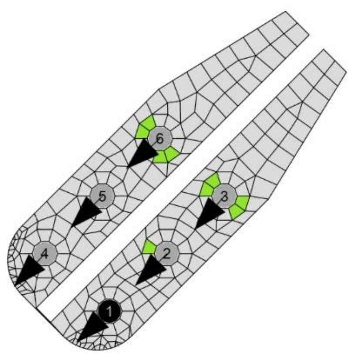
In the CBFEM framework implemented in IDEA StatiCa, bolts are modeled as nonlinear spring elements that link connected plate components. Each bolt is characterized by directional stiffness values—separately defined for shear and axial behavior—and its load-deformation response is continuously monitored during analysis. The model incorporates important geometric and detailing factors, including bolt diameter, layout, pretensioning, and edge distances.

Shear demand is computed for each bolt individually, and the critical bolt is identified by comparing the applied force to its nominal shear capacity. In bolted groups—such as those typically found in gusset or lug connections—each bolt is evaluated independently in both CBFEM simulation output and analytical calculations. The overall connection capacity is then governed by the most highly utilized bolt, ensuring that localized overstress or premature bolt failure is not overlooked.

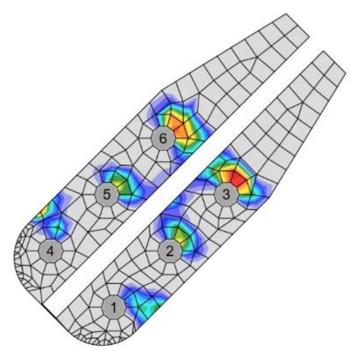
This rigorous evaluation approach—combining code-based interaction checks with detailed CBFEM modeling—ensures that bolt shear capacity is adequately addressed in the seismic performance of BRBF connections, particularly under load-reversal and cyclic demand conditions.



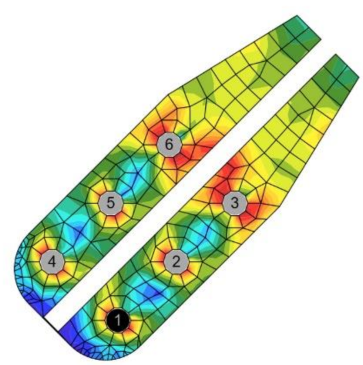
a) Evaluation of bracing connection for limit state of bolt shear - CBFEM



b) Bolt forces for strain check
-CBFEM



c) Stresses in contact
-CBFEM



d) von Mises Stresses in Lug Plate
-CBFEM

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 50.71 kip ≥ $V =$ 28.42 kip

Where:

$F_{nv} = 68.0$ ksi – nominal shear stress AISC 360-16 – Table J3.2

$A_b = 0.994$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Figure 18 – Evaluation of Limit State of Bolt Shear in Connection – CBFEM

In BRBFs, bolts between the lug plate and gusset plate are especially critical in transferring axial forces from the brace. The lug plate transmits this force into the gusset via shear in the bolts as shown in Figure 17.

In IDEA StatiCa, this load path is simulated based on the actual geometry and bolt layout, so the shear force is not assumed to be equally distributed—it reflects plate deformation and stiffness. Each bolt's internal force is extracted from the spring element, which represents its mechanical response:

$$V_{bolt}=\text{shear force in the nonlinear spring (per bolt)}$$

As per AISC 360-16, Section J3.6, the nominal shear strength of a bolt is:

$$R_n=F_{nv}A_b$$

Where:

- R_n is Nominal shear strength per shear plane.
- F_{nv} is Nominal shear strength per bolt.
- A_b is Nominal bolt area.

Design strength (LRFD):

$$\phi R_n= \phi F_{nv}A_b N_{shear\ planes}$$

Where:

- Φ is Resistance factor = 0.75 for shear in bolts (per Table J3.1)
- $N_{shear\ planes}$ is either 1 or 2 , depending on the number of shear planes.

Utilization in CBFEM for bolts calculated as

$$U_{bolts} = V_{bolt} / \phi R_n$$

As illustrated in Figure 17, the comparison between analytical (AISC-based) and numerical (CBFEM) results for bolt shear capacity reveals excellent agreement. The bolt shear force computed using AISC 360-16 provisions was found to be 50.6 kips, while the CBFEM simulation in IDEA StatiCa predicted a nearly identical value of 50.7 kips for the most critically loaded bolt. This agreement occurs at the axial brace force of 345 kips, which corresponds to 100% bolt utilization in the CBFEM model.

Such close correlation not only validates the accuracy of the CBFEM approach in capturing bolt behavior under high axial seismic demands but also confirms the reliability of AISC equations for preliminary design checks. It reinforces the suitability of using traditional analytical methods during initial design stages, while advanced finite element modeling like CBFEM can be employed for detailed performance verification and capacity confirmation under capacity-based seismic loading.

10.d. Bolt Bearing

In Buckling-Restrained Braced Frame (BRBF) systems utilizing bolted lug connections, evaluating the bolt bearing limit state is critical—especially at interfaces where axial forces are transferred from the lug plate to the gusset plate. Bolt bearing failure occurs due to localized compressive stresses that cause deformation or tearing around the bolt hole in the connected plate.

When two plates are joined using bolts (e.g., lug to gusset), each plate must be assessed separately for bearing resistance. The bearing capacity of each component depends on key factors such as plate thickness, edge distance, bolt diameter, and the material's yield and ultimate strengths. Since these properties may differ between the gusset and lug plates, bearing checks must be conducted independently for both.

Design criteria for bolt bearing strength are outlined in Section J3.6 of AISC 360-16, which provides equations to prevent excessive deformation or rupture under localized compressive stress. Compliance with these provisions is essential in seismic applications to ensure that localized bearing failure does not compromise the connection's performance, especially under cyclic or overstrength-level axial load.

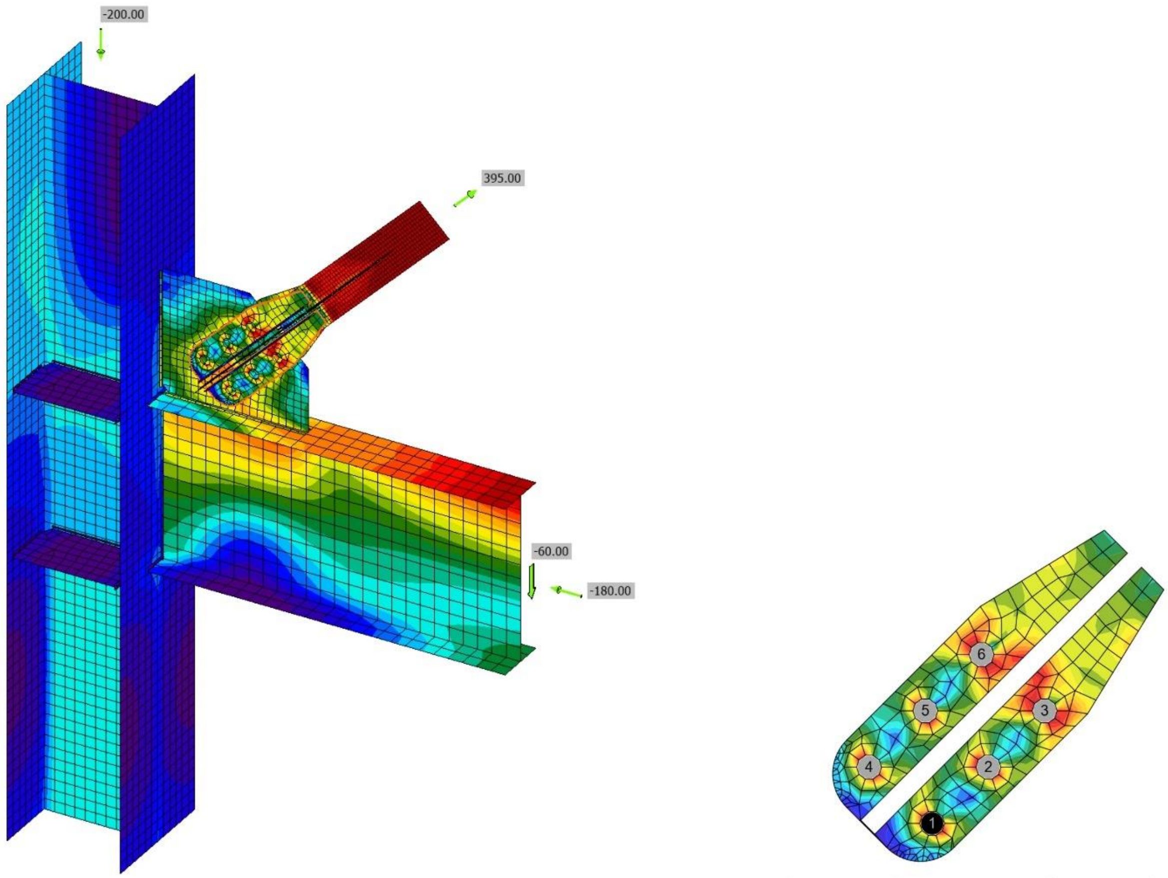
$$\phi R_n = \phi min(1.2 L_c t F_u,\ 2.4 d t F_u)$$

Where:

- ϕ = resistance factor (usually 0.75 for LRFD)
- L_c is clear distance in load direction between the edge of the hole and the edge of the plate (in.)
- t is plate thickness (in.)
- d is nominal bolt diameter (in.)
- F_u is ultimate strength of the plate material (ksi)

Within IDEA StatiCa Connection, the bolt bearing evaluation is seamlessly incorporated into the "Bolt Check" module. The software automatically performs bearing strength assessments by considering parameters such as bolt and hole type, spacing (pitch), and edge distances in accordance with AISC 360-16 provisions. When the bolts are subjected to combined shear and tension, this interaction is also taken into account in the bearing check.

For each bolt in the connection, individual bearing utilization ratios are computed and displayed, enabling precise identification of critical bolts within the lug-to-gusset interface. This functionality ensures that the bearing strength requirements are met for all connection components, thus providing a reliable verification against local failure modes and supporting robust seismic design practices.



a) Evaluation of bracing connection for limit state of bolt bearing - CBFEM

b) von Mises Stresses in Lug Plate -CBFEM

Bearing resistance check (AISC 360-16 – J3-6, AISC 360-16 – J7-1)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 1.80 \cdot F_y \cdot A_{pb}$$

$$\phi R_n = \quad 61.05 \text{ kip} \quad < \quad V = \quad 61.61 \text{ kip}$$

Where:

$l_c = 1.04$ in	– clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
$t = 1.00$ in	– thickness of the plate
$d = 1.13$ in	– diameter of a bolt
$F_u = 65.0$ ksi	– specified minimum tensile strength of the connected material
$F_y = 50.0$ ksi	– specified minimum yield stress
$A_{pb} = d \cdot t = 1.13$ in^2	– projected area in bearing
$\phi = 0.75$	– resistance factor for bearing at bolt holes

Figure 19 – Evaluation of Limit State of Bolt Bearing in Connection – CBFEM

The bolt shear capacity calculated using the AISC provisions (61.05 kips) closely matches the result obtained from the CBFEM analysis in IDEA StatiCa (61.05 kips) for the most critically loaded bolts. At this load level, the bolt utilization reaches 100% under an applied axial tension load of 395 kips, confirming strong agreement between analytical and numerical methods, as illustrated in Figure 19.

10.e. Bolt Tearout

Bolt tearout, also known as edge rupture, is a critical failure mode that must be carefully evaluated in bolted corner BRB (Buckling-Restrained Braced Frame) connections, particularly at the lug-to-gusset interface. This type of failure occurs when inadequate edge distance permits the bolt to tear a strip of material from the plate under axial loading. The risk is most pronounced in two key components:

- Lug plate: This component directly transmits axial force from the BRB to the gusset and is subjected to high stress concentrations near the bolt holes.
- Gusset plate: Often thinner than the lug plate, the gusset is especially susceptible to edge rupture—particularly when subjected to high brace forces, reduced bolt spacing, or tight edge distances.

According to AISC 360-16, Section J4.3, tearout failure—or edge rupture—occurs when the localized tensile stresses at the bolt hole exceed the edge strength of the connected plate. This limit state is especially critical in seismic connections such as Corner BRB joints, where high axial loads from the brace are transferred through bolts.

The design edge rupture strength is evaluated using the following equation from AISC 360-16:

$$\phi R_n = \phi 0.6 F_u l_e t$$

Where:

- F_u is Ultimate strength of the plate (lug or gusset)
- l_e is Edge distance in the load direction
- t is Plate thickness
- ϕ is 0.75

Unlike bolt bearing, which involves localized deformation around bolt holes, bolt tearout is a brittle failure mode characterized by sudden edge rupture. This makes it critical to evaluate both bearing and tearout limit states independently for each connected plate in bolted BRB connections.

In IDEA StatiCa Connection, bolt tearout is not explicitly assessed as a distinct failure mode. The software does not automatically flag edge rupture failures; therefore, we must manually verify compliance with AISC 360-16 Table J3.4 for minimum edge distances.

To mitigate the risk of tearout and ensure design adequacy, the following best practices are recommended:

- Visually inspect IDEA StatiCa results for plastic strain concentrations near bolt holes, especially along plate edges.
- Review bolt utilization ratios in the output; values exceeding 90% near edge locations warrant closer scrutiny and possible redesign.

While IDEA StatiCa provides valuable stress and strain visualizations, manual verification using AISC provisions remains essential to confirm that edge rupture does not govern the connection design. In this study, CBFEM results aligned with AISC-based checks, affirming the safety of the bolted lug-to-gusset connection against tearout failure.

10.f. Welded Connections

Welds play a critical role in the force transfer mechanism of Buckling-Restrained Braced Frame (BRBF) systems, particularly at high-demand regions such as lug-to-gusset, gusset-to-beam, gusset-to-stiffener, and gusset-to-column interfaces. Their design must ensure adequate capacity to resist axial, shear, and bending forces without premature failure. In seismic applications, welds are also required to withstand cyclic loading, where fatigue resistance, ductility, and detailing quality are paramount. For essential components like lug plates, shop welds are preferred due to superior quality control, while field welds, when necessary, must be subject to rigorous inspection and assurance protocols.

From an analysis standpoint, the Component-Based Finite Element Method (CBFEM) offers a more advanced and realistic approach compared to conventional AISC 360-16 weld design provisions. While traditional methods often simplify assumptions—neglecting eccentricities, torsional effects, deformation compatibility, and Poisson effects—CBFEM models these behaviors explicitly. Using a 3D finite element mesh, the weld geometry, material behavior, and nonlinear stress distribution are fully captured. Welds are represented with elastic-plastic response models, where yielding is controlled by stress states across the effective throat area.

In IDEA StatiCa, weld checks are performed using computed normal and shear stresses acting on the weld throat ($\sigma_{\perp}$, $\tau_{\perp}$, $\tau_{\parallel}$). These are used to derive the resultant force (F_n) and inclination angle (ϕ), which reflect the orientation of the load and the connection configuration. Weld utilization ratios are then calculated and reported at each critical location, enabling verification of capacity under seismic-level axial demands. It is common for utilization to reach or approach 100% at brace interfaces under maximum design loads.

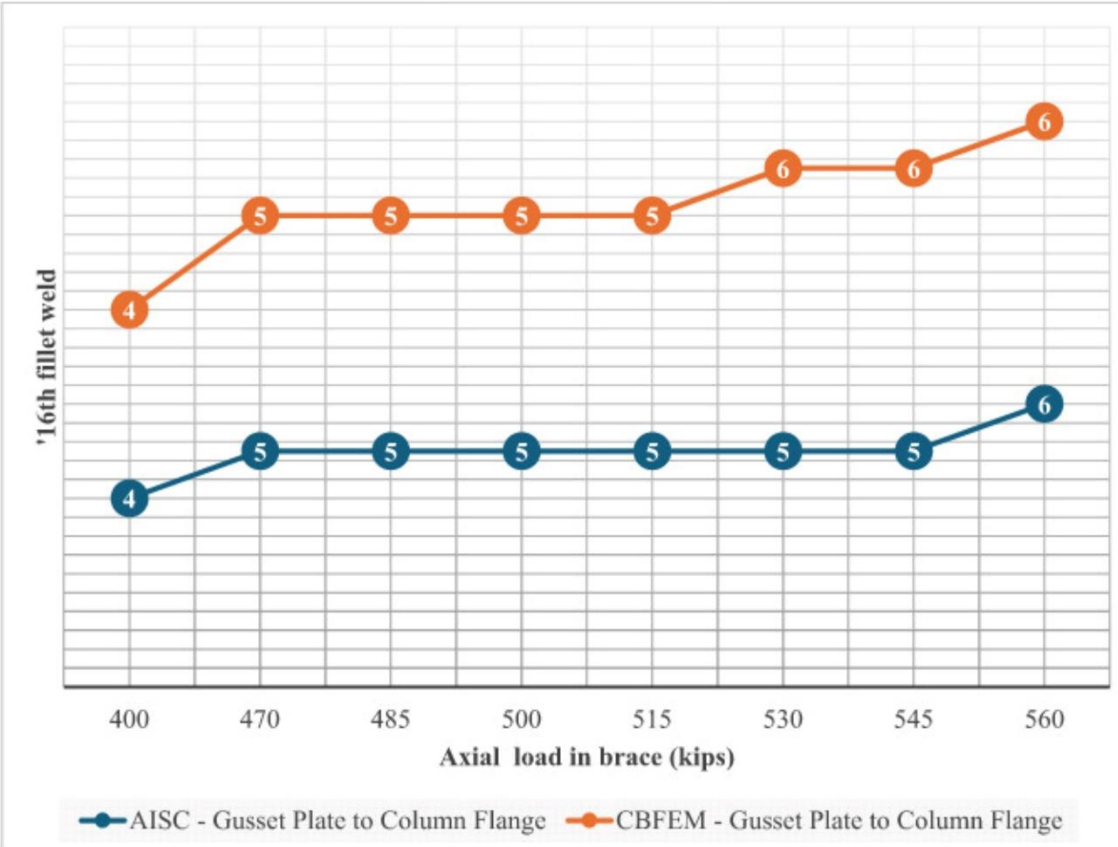
Compared to AISC-based calculations, CBFEM may require slightly larger weld sizes. This results from its ability to account for local stress redistributions within weld groups, thereby improving accuracy but also indicating a need for increased weld robustness. This tendency is confirmed in Table 3 and the corresponding stress and strain distributions in Figures 20, which show higher utilization in regions of localized demand.

Despite the more conservative sizing recommendations, the results from CBFEM align well with AISC estimates, reinforcing the method’s suitability for performance-based seismic design of welded BRBF connections.

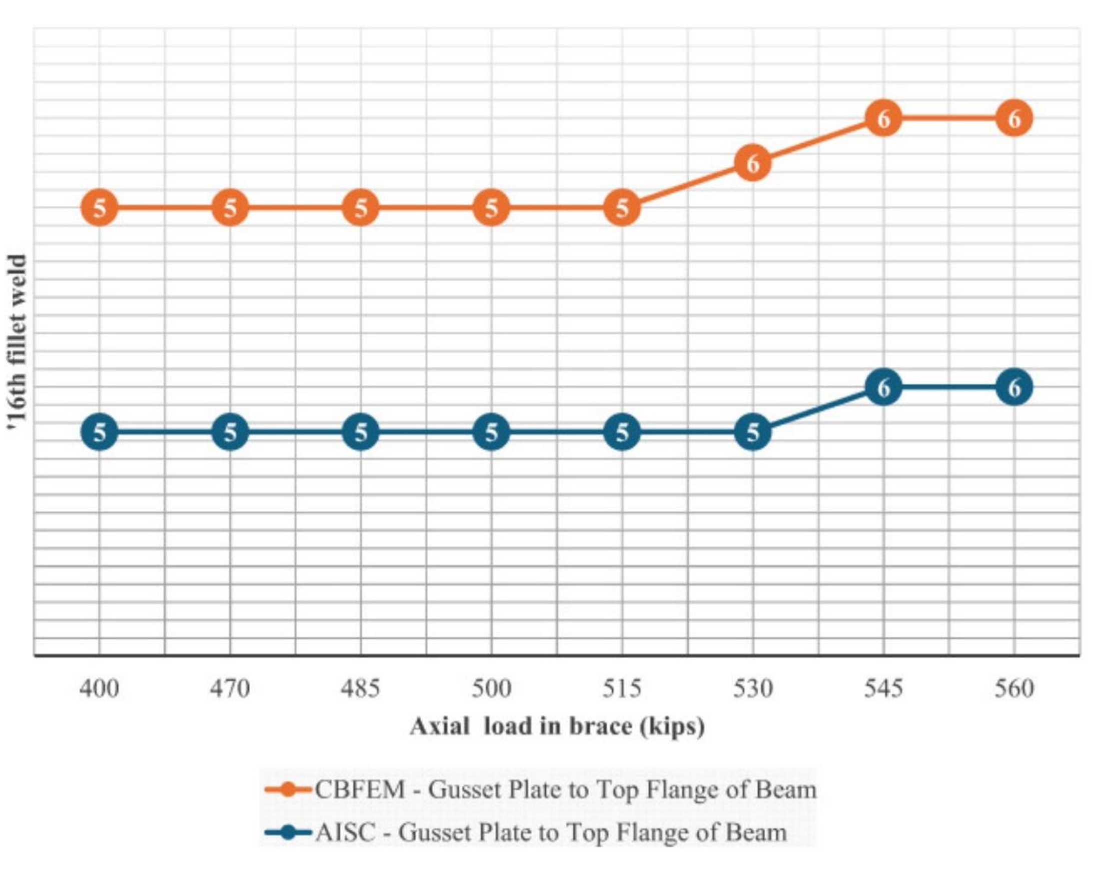
Sr No.	P_{brace} (kips)	Check of weld connecting	Required '16ths weld size (in)	
			AISC	CBFEM
1	505	Gusset plate to beam top flange	5/16	5/16
2	530	Gusset plate to column	5/16	5/16
3	555	Gusset plate to lug plate	3/8	3/8

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

Table 3: Comparison of fillet weld size as per AISC and CBFEM



a) Fillet weld connecting gusset plate to column flange



b) Fillet weld connecting gusset plate to top flange of beam

Figure 20 – Comparison of Required Fillet Weld size for Corner BRB Connection at Two Locations

Figure 20 illustrates a comparative analysis of fillet weld sizing requirements at two key locations in a Corner Buckling-Restrained Brace (BRB) connection—namely:

1. Gusset plate to stiffener plate, and
2. Gusset plate to beam flange.

The weld sizes are determined under increasing axial brace loads, ranging from moderate (~400 kips) to high seismic demand levels (~560 kips), and are compared using two different methodologies:

- AISC 360-16 Provisions: This traditional approach calculates required weld size based on the assumption of uniform stress transfer, elastic material behavior, and simplified load distribution models. The weld design is governed by the demand at the throat of the weld and uses nominal strength equations with safety factors applied.
- CBFEM in IDEA StatiCa: The Component-Based Finite Element Method models the actual weld geometry, stiffness, and inelastic material behavior, capturing non-uniform stress distributions, eccentricities, and plastic strain development at the weld throat. It includes 3D interaction between connected elements, force redistribution, and more precise modeling of real load paths.

Observations from Figure 20:

- At lower to intermediate axial brace loads (e.g., 400–500 kips), both AISC and CBFEM methods recommend similar fillet weld sizes at the studied locations—typically between 1/4" to 5/16". This reflects a convergence between traditional design conservatism and the inelastic reserve capacity captured in CBFEM.
- As axial forces increase toward 560 kips, the CBFEM approach begins to recommend larger weld sizes—up to 3/8"—particularly at the gusset-to-beam flange interface. This is due to the localized stress concentrations observed in the finite element analysis, which are not reflected in AISC's linear assumption models.
- The discrepancy between AISC and CBFEM grows at higher loads because CBFEM captures the actual behavior of the weld under seismic loading, including plasticity, complex load paths, and eccentric force applications. This results in a more performance-based evaluation that is especially important in seismic connections, where welds must remain ductile and fatigue-resistant.

This comparison emphasizes the importance of supplementing traditional code checks with advanced analysis tools like CBFEM in high-demand seismic applications. While AISC provisions remain conservative and useful for initial sizing, CBFEM provides a more accurate reflection of real-world performance, especially under large cyclic loads. The figure underscores the need for performance-based verification in connections involving BRBs, where weld failure can compromise the entire energy dissipation mechanism.

Furthermore, it highlights that CBFEM can identify potential overstress or inadequate weld sizing that may not be evident in hand calculations, making it a valuable design validation tool in modern seismic engineering practice.

11. Evaluation of Buckling for Compression Load in Brace – CBFEM

In Buckling-Restrained Braced Frames (BRBFs), the defining innovation lies in the brace's ability to yield in both tension and compression without undergoing global buckling. This is achieved using a steel core encased in a buckling-restraining mechanism, which transforms the brace into a highly ductile energy-dissipating component. However, for the system to function as intended during seismic events, all non-dissipative elements—including gusset plates, beam flanges, column webs, and stiffeners—must remain elastic throughout the inelastic response of the BRB core.

Among these, the gusset plate is particularly vulnerable to local out-of-plane buckling during compressive loading. While the BRB core is effectively restrained against global instability, the gusset remains susceptible to localized plate buckling, which could compromise the brace's ability to dissipate energy and violate the capacity design principles defined in AISC 341-16. To mitigate this, (Dowswell, Bo, 2006) introduced a mechanics-based compactness criterion, which models the gusset as a cantilevered plate subjected to axial compression.

The gusset plate slenderness is evaluated using the non-dimensional parameter:

$$\lambda_c = (L_e / t) \, .\sqrt{(F_y / E)}$$

Where:

- λ_c = Slenderness parameter for gusset compactness
- L_e = Effective length of the gusset plate in inch
- t = Thickness of the gusset plate in inch
- F_y = Yield strength of the gusset material in ksi
- E = Modulus of elasticity of steel (typically 29,000 ksi)

A compact gusset is defined by satisfying the condition

$$\lambda_c \leq \lambda_{c,limit}$$

where the limiting value ensures elastic behavior under expected brace loads.

To further ensure elastic behavior, the gusset plate must be checked against the brace's probable compressive strength:

$$P_b = \omega . R_y . F_{y,BRB} . A_{core}$$

Where,

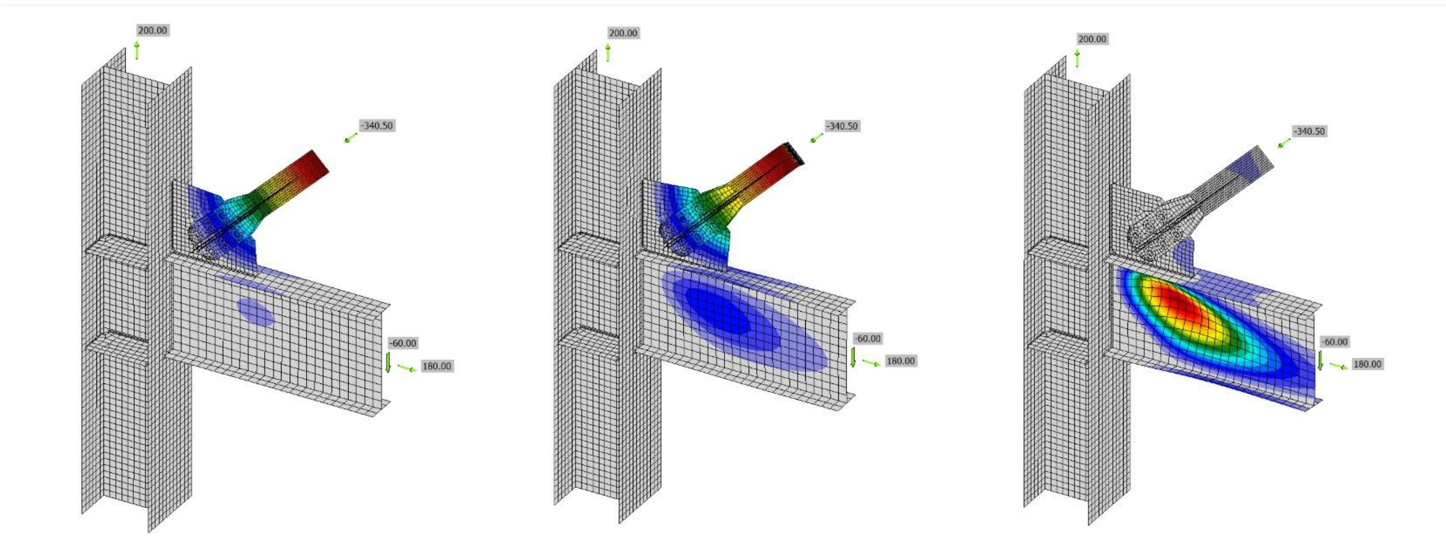
- ω = Overstrength factor,
- R_y= Material strain-hardening factor (typically ~1.1–1.2),
- $F_{y,BRB}$ = Yield strength of steel core in ksi
- A_{core} = Area of the BRB steel core.

Bo Dowswell's slenderness approach has been integrated into Appendix D of AISC 341-16, which prescribes:

- Minimum effective gusset lengths
- Fillet radius and edge distance limits
- Elastic design requirements under expected axial loads

In the current study, the gusset plate for the corner BRB connection was detailed to meet compactness requirements by:

- Minimizing Effective Length - L_e via geometric alignment with brace work points
- Using 1-inch thick gusset plates for increased stiffness
- Employing double-sided lug plates to enhance restraint
- Evaluating out-of-plane instability modes using CBFEM in IDEA StatiCa



a) Mode shape 1 for Buckling Analysis of Corner BRB Connection - CBFEM

b) Mode shape 2 for Buckling Analysis of Corner BRB Connection - CBFEM

c) Mode shape 3 for Buckling Analysis of Corner BRB Connection - CBFEM

Figure 21 – Buckling Analysis of Corner CoreBrace BRB connection - CBFEM

Figure 21 illustrates three buckling mode shapes obtained from a Linear Buckling Analysis (LBA) using CBFEM:

- Mode Shape 1, also referred as critical buckling mode of connection reveals local buckling near the brace-gusset interface, correlating with AISC 360-16 Section E7 and J4 for connection element buckling.
- Mode Shape 2 shows interaction-induced instability in the gusset and beam web panel, aligned with AISC 341-16 Section F2.6c.
- Mode Shape 3 depicts crippling of the beam web panel due to bearing stresses from gusset loading, governed by AISC 360-16 Sections J10.2 and J10.3.

12. Evaluation of Limit States Related to Members – CBFEM

12.a. Web Local Yielding of Beam Member

Web local yielding is an important limit state that must be evaluated when designing corner BRB connections, particularly at the location where concentrated loads from gusset or lug plates are transferred into the web of the beam. This failure mode occurs when the applied bearing stresses exceed the yield strength of the web material, resulting in localized plastic deformation. Such yielding may compromise the connection's ability to safely transfer brace forces, especially under compression, where BRBs exert higher loads. Since Buckling-Restrained Braces are specifically engineered to yield in both tension and compression, the surrounding frame components—including the beam web—must remain elastic to ensure proper energy dissipation occurs solely within the brace core.

According to AISC 360-16, web local yielding is evaluated using the provisions of Section J10.2. These provisions provide formulas for computing the maximum force that can be applied to a beam web through a concentrated load without causing yielding. The key design parameters influencing this limit state include the web thickness, yield strength of the beam material, and the bearing length (i.e., the length over which the load is applied). While the AISC approach assumes a discrete bearing length and uniform force application, the Component-Based Finite Element Method (CBFEM) used in IDEA StatiCa models this behavior with greater granularity.

In IDEA StatiCa, stresses and plastic strains are distributed over shell elements, which may mask localized effects and make it difficult to isolate the precise onset of web yielding. Furthermore, the software does not provide a direct code check or pass/fail indicator for web local yielding, unlike other components such as bolts, welds, or gusset plates. This presents a challenge for us attempting to validate this limit state solely through CBFEM results.

To overcome these limitations, we must interpret IDEA StatiCa's visual results—especially plastic strain distributions and von Mises stress contours—at the beam web near the gusset interface. Zones with elevated plastic strain near the gusset-bearing area may indicate potential yielding. These results should be cross-verified with manual calculations using AISC expressions to ensure that the beam web remains within elastic limits. Additionally, where high stress concentrations are observed, the use of transverse stiffeners or extended bearing lengths can help mitigate the risk of web local yielding.

Overall, for the presented example web local yielding of beam member was evaluated using AISC equation manually and was observed to not be a governing limit state.

12.b. Web Local Crippling of Beam Member

Web local crippling is a potential failure mode that must be considered in corner Buckling-Restrained Brace (BRB) connections, particularly at locations where concentrated compressive loads from gusset or stiffener plates are transferred into the beam flanges and web. Unlike web local yielding, which pertains to uniform compression over a bearing length, web crippling refers to localized crushing or instability of the web adjacent to the point of load application—often occurring without stiffeners or where the load is applied close to the web-flange junction.

In the context of Corner BRB connections, the brace force—especially under compression—can induce a significant localized force through the gusset or lug into the beam flange and subsequently into the web. If no transverse stiffeners are provided, the web must resist these forces alone, making it vulnerable to crippling. This is particularly critical for short bearing lengths, thin webs, or when the load is applied near the flange without sufficient support.

According to AISC 360-16, web local crippling is addressed in Section J10.5, which provides design strength limits for webs subjected to concentrated forces. The allowable strength depends on factors such as web thickness (t_w), web depth, inside bend radius, distance to adjacent stiffeners or load points, and whether the load is applied at the flange or within the web.

In IDEA StatiCa, web local crippling is not explicitly checked using the J10.5 equations. However, the CBFEM framework allows us to observe stress concentrations and plastic strain zones near the gusset-beam interface. In cases where crippling is likely, high compressive stresses or localized plasticity adjacent to the beam flange-web junction may be visible in the stress or strain contour plots. Still, interpreting these results requires engineering judgment, since IDEA StatiCa does not directly indicate failure modes like local crippling.

To mitigate this risk, we should:

- Add transverse stiffeners near gusset-to-beam interfaces.
- Ensure sufficient bearing length to spread the load over a wider area of the web.
- Verify edge distances and location of load application per AISC guidelines.
- Cross-check with hand calculations using AISC 360-16 Section J10.5 to validate the web's capacity against crippling.

Overall, for the presented example web local crippling of beam member was evaluated using AISC equation manually and was observed to be not a governing limit state.

12.c. Compressive Strength of Brace Member

The primary scope of IDEA StatiCa Connection includes checking bolts, welds, plates, and local stress and strain behavior at joint interfaces. However, when it comes to evaluating the member-level behavior of braces—particularly in Buckling-Restrained Braced Frames (BRBFs)—the tool has clear limitations.
Most notably, IDEA StatiCa does not account for:

- Global buckling behavior of the brace member,
- Inelastic yielding along the BRB core, and
- Transition zone effects, which are crucial in BRB design.

These phenomena are inherently part of the brace as a full structural member, and their evaluation requires tools that model the entire brace length, including casing effects, boundary conditions, and member slenderness. As such, brace compressive strength—which is governed not by buckling but by controlled yielding in the steel core—cannot be realistically assessed using IDEA StatiCa Connection alone.
The compressive strength of a BRB depends on:

- The core cross-sectional area (Asc),
- Material properties such as yield strength (Fy) and strain hardening characteristics (ω), and
- The confining effectiveness of the brace casing, which prevents buckling of the steel core and ensures uniform axial yielding.

These parameters are outside the modeling capability of IDEA StatiCa, which does not simulate the full inelastic behavior of BRBs, including post-yield deformation capacity or casing confinement. Additionally, transition zones—where the yielding core meets the stiffer non-yielding regions—are not modeled in the connection module.

In the CBFEM framework, member ends are idealized with simple boundary conditions (e.g., pinned or bearing), and the system does not model the full-length structural behavior, such as:

- Slenderness effects,
- Global instability modes, or
- Frame-system interactions that influence brace axial capacity under compression.

Consequently, IDEA StatiCa provides local joint response, not global member behavior, making it inappropriate for evaluating axial compression capacity of BRBs beyond the connection zone. For complete assessment, we must rely on:

- Member design tools (e.g., hand calculations, OpenSees, SAP2000),
- Manufacturer test data, and
- Guidelines per AISC 341-16 and AISC 360-16

12.d. Column Web Local Yielding

Column web local yielding is a critical limit state that must be checked in Buckling-Restrained Braced Frame (BRBF) corner connections, particularly where concentrated forces from the brace or gusset plate are transmitted into the web of the column. This condition arises when the bearing pressure from the gusset plate or stiffener exceeds the yield strength of the column web material over a localized area, resulting in permanent deformation. In BRBF systems, where braces are designed to yield in both tension and compression, these concentrated loads at the gusset-column interface can become substantial during seismic events.

According to AISC 360-16 Section J10.2, web local yielding should be assessed using a formula that considers the web thickness, yield strength (Fy) of the column, and the bearing length of the loaded area. If a transverse stiffener is not present, the column web may become the weakest link, especially under high axial or diagonal loads transferred through the gusset plate. The required check ensures that the bearing stress in the web does not exceed the allowable limit, preventing plastic indentation or collapse of the web region.

In IDEA StatiCa, evaluation of column web local yielding can be performed indirectly through plastic strain visualization and stress contours in the finite element mesh. While the software does not provide a direct utilization ratio for this limit state, we can interpret:

- Localized high von Mises stresses, or
- Plastic strains exceeding 2–3% in the column web adjacent to the gusset as indicators of potential web yielding. If yielding is observed near weld toes, gusset corners, or bearing regions, then we should assess whether the affected area is within the acceptable range based on expected inelastic performance.

If local yielding appears critical, the following measures can help mitigate it:

- Adding transverse stiffeners to reinforce the web
- Increasing the gusset bearing length on the web
- Using a heavier column section with thicker web
- Distributing loads through additional plates or stiffeners to reduce local pressure

Overall, for the presented example, column web local yielding was evaluated using the AISC equation manually and was observed to be not a governing limit state.

12.e. Column Web Crippling

Column web crippling is a localized failure phenomenon that arises when a concentrated compressive force is transferred transversely into the web of a column, typically without adequate stiffening. In the context of corner Buckling-Restrained Braced Frame (BRBF) connections, this occurs at the interface where a gusset plate or stiffener connects to the column web or flange. Since BRBF systems are designed to dissipate seismic energy through inelastic deformation of the brace core, the axial loads transferred from the brace into the column can be significant—particularly during design-level earthquake events. When these concentrated loads are applied to slender webs without sufficient bearing length or transverse reinforcement, local instability or crushing can develop, leading to a crippling failure mode.

According to AISC 360-16, column web crippling should be evaluated using the provisions in Section J10.3, which provide empirical formulas to calculate the crippling capacity of unstiffened webs under concentrated forces. These equations consider factors such as web thickness, yield strength, bearing length, edge distance, and whether or not transverse stiffeners are provided. The objective is to ensure that the column web can safely transfer the applied brace forces without exhibiting excessive local deformation or collapse.

In IDEA StatiCa, which uses the Component-Based Finite Element Method (CBFEM), column web crippling is not directly reported as a distinct limit state. Instead, the software evaluates stress and strain distribution within the joint components, including the column web, by using shell elements and nonlinear material models. Stress concentration and plastic strain contours can provide indirect evidence of crippling potential. For instance:

- If von Mises stress values approach or exceed the material yield strength near the gusset-column interface, or
- If localized plastic strain exceeds 3–5%, it may suggest that the web is undergoing distress that could be associated with crippling.

However, since the finite element mesh smooths force transfer across a region rather than concentrating it at a discrete point, IDEA StatiCa may underrepresent the severity of web crippling. This presents a challenge, especially in seismic systems where precision is critical. We should supplement the CBFEM results with hand calculations based on AISC equations for web crippling, particularly for W-shapes with thin webs or when transverse stiffeners are not provided. Furthermore, attention should be given to detailing—ensuring adequate bearing length, verifying minimum edge distances, and introducing web stiffeners where needed.

Overall, for the presented example, column web crippling was evaluated using the AISC equation manually and was observed to be not a governing limit state.

13. Comparison of Results – AISC vs CBFEM

1. Evaluation of Limit State for Tension Load in Brace				
Sr No	**Limit State of**	**AISC (kips)**	**CBFEM (kips)**	**Δ_{AISC_CBFEM} (%)**
1	Tensile Yielding of Gusset Plate	450	476	6%
2	Block Shear Rupture of Gusset Plate	480	485	1%
3	Bolt Shear	51	51	0%
4	Bolt Bearing	61	61	0%
5	Bolt Tearout	OK	OK	N.A

2. Comparison of weld sizes as per AISC and CBFEM				
Sr No.	**P_{brace} (kips)**	**Check of weld connecting**	**Required ‘16^{ths} weld size (in)**	
			AISC	**CBFEM**
1	505	Gusset plate to beam top flange	5/16	5/16
2	530	Gusset plate to column	5/16	5/16
3	555	Gusset plate to lug plate	3/8	3/8

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

14. Conclusion

This study provided a comprehensive evaluation of a bolted CoreBrace Corner Buckling-Restrained Brace (BRB) connection using the Component-Based Finite Element Method (CBFEM) in IDEA StatiCa V25.0, benchmarked against traditional analytical methods outlined in AISC 360-16 and AISC 341-16. The objective was to verify the seismic adequacy of the connection, particularly in the context of capacity design principles, and to demonstrate the effectiveness of CBFEM in visualizing stress and strain distributions in seismicbracing systems.

The connection modeled was a vertical bracing configuration located at a corner, characterized by high force demand, and critical interaction zones between the brace, gusset, beam, and column. The BRB was modeled to reflect its expected yielding behavior under seismic loads, while all surrounding elements—including gusset plates, bolts, stiffeners, and framing members—were intended to remain elastic per seismic capacity design requirements.

The analysis explored both axial tension and axial compression loading scenarios, revealing distinct behavioral patterns. Under tension, the gusset plate performed elastically up to 300 kips and exhibited localized yielding between 400 and 500 kips. Plastic strain concentrations developed around bolt holes and gusset edges, yet remained within acceptable limits, confirming that the gusset detailing was effective for the expected seismic loading. Under compression, the gusset behavior was more complex, showing asymmetric stress propagation, widespread plastic strain, and potential out-of-plane deformation—validating the need for manual evaluation of gusset compactness and robust detailing under higher compressive demand.

CBFEM results were cross-verified with hand calculations for multiple limit states, including bolt shear, bolt bearing, weld strength, block shear, bolt tear-out, web local yielding, and web crippling. The agreement between IDEA StatiCa outputs and AISC equations confirmed the software's reliability for connection-level evaluation. While weld utilization ratios were slightly more conservative in CBFEM, this was attributed to its ability to capture real stress redistribution patterns not accounted for in simplified methods.

In addition, a Linear Buckling Analysis (LBA) using CBFEM identified the first three critical buckling mode shapes: local buckling at the gusset-to-brace interface, interaction-induced deformation in the beam web panel, and crippling of the beam web. These mode shapes correlated directly with provisions in AISC 360-16 (Sections E7, J10.2, and J10.3) and AISC 341-16 (Chapter F and Appendix D).

Special emphasis was placed on the gusset plate design per Bo Dowswell's compactness criteria. The study confirmed that the gusset plate slenderness ratio (λ_c) met the compactness limits required to ensure elastic performance under the BRB's probable compressive strength. These calculations, coupled with strain contour analysis in IDEA StatiCa, demonstrated that the gusset was sufficiently restrained against premature out-of-plane buckling.

Overall, the study demonstrated that the evaluated corner BRB connection, when detailed per AISC 341-16 and verified using CBFEM, meets seismic performance requirements. The method provided clear insights into force transfer mechanisms, strain development, and potential instability regions. IDEA StatiCa's visualization capabilities—particularly for plastic strain and buckling—enabled the identification of critical detailing improvements such as increasing gusset thickness, optimizing bolt layout, and adding web stiffeners.

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