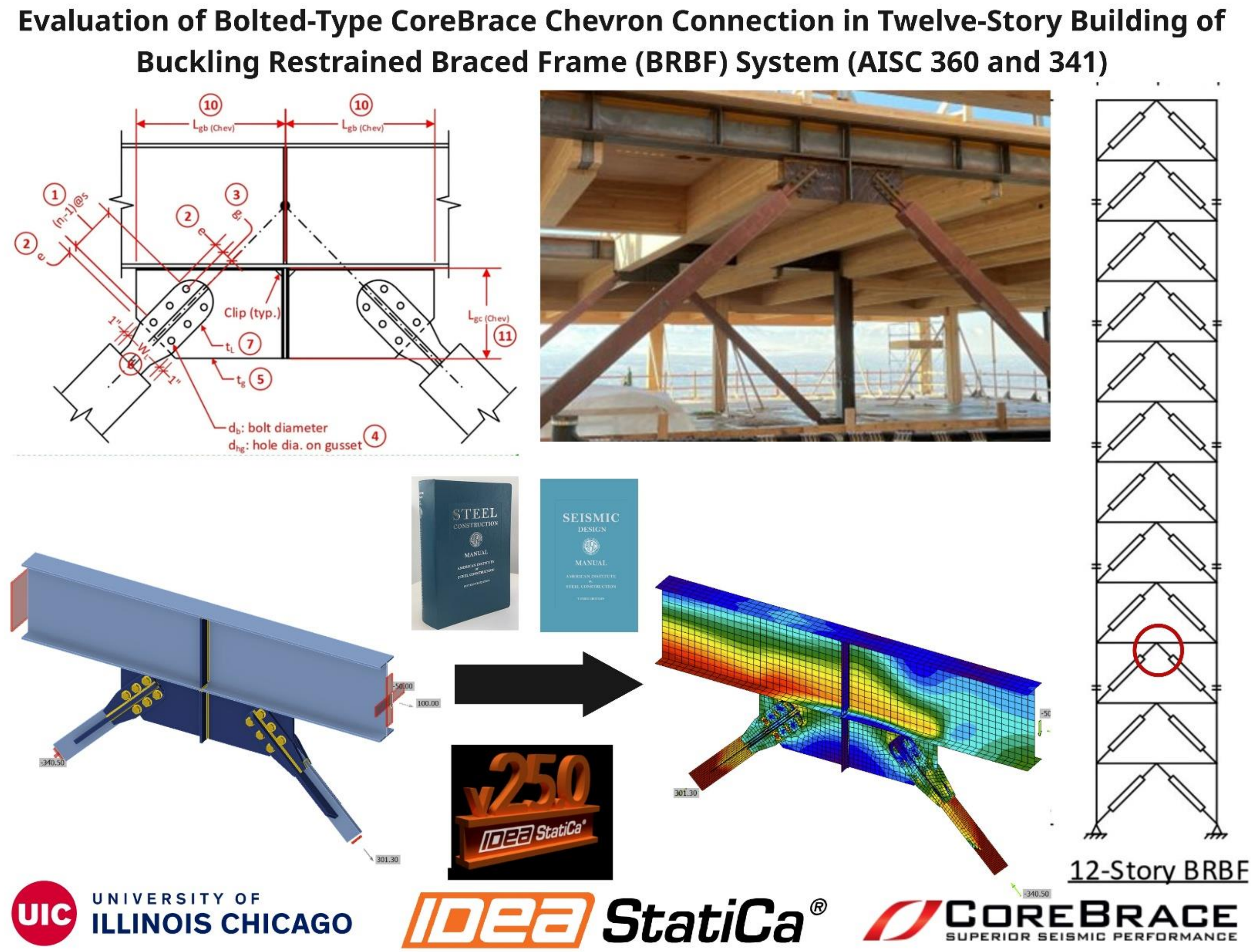

Cover Image

This report presents the first 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 Chevron 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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1. Chevron Bracing Connections in BRBF

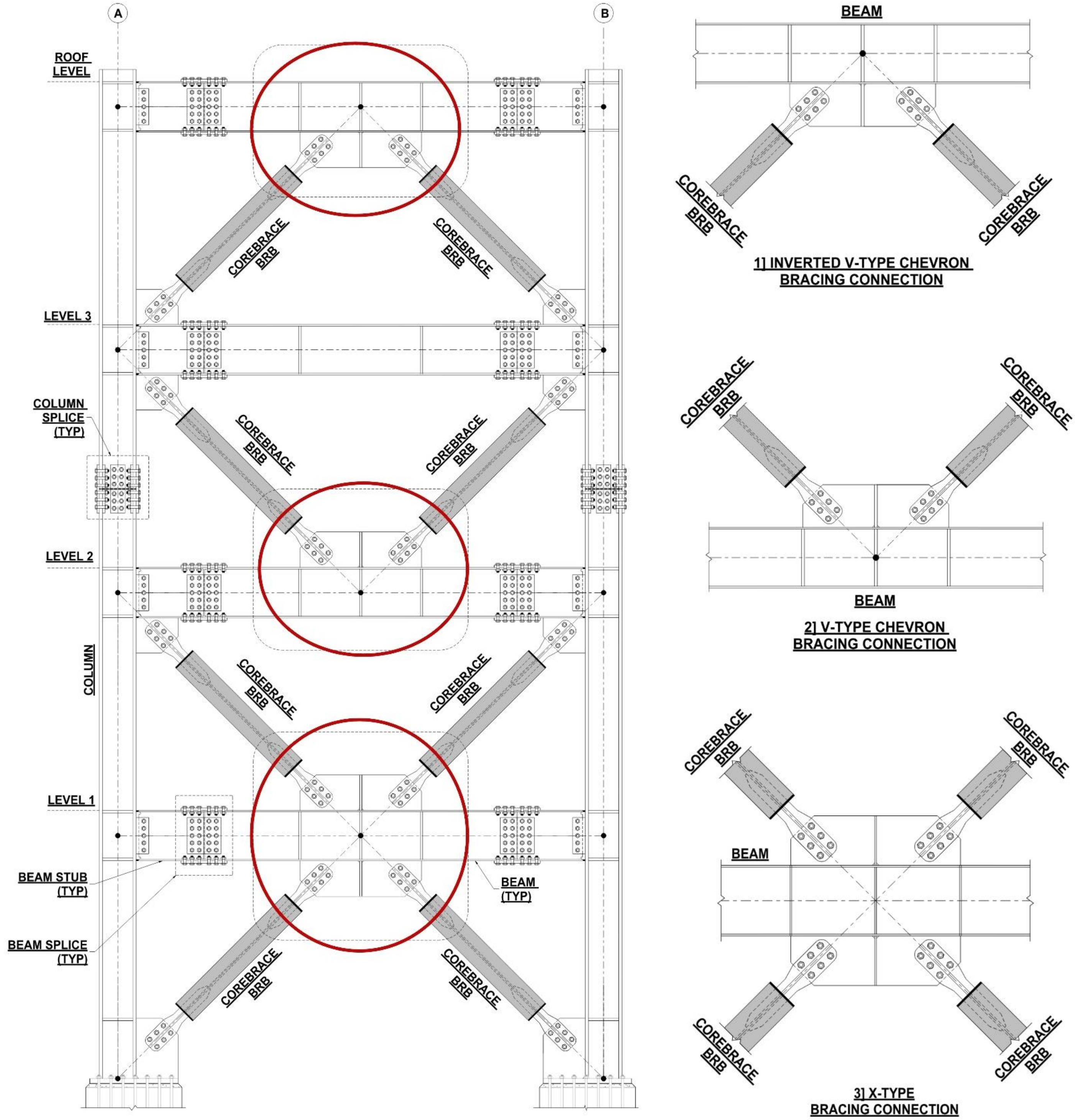


Figure 1 – Configurations of Chevron Connections in Buckling-Restrained Braced Frames (BRBFs)

Chevron bracing configurations are widely adopted in steel-framed buildings as an effective means of providing lateral-load resistance. These systems comprise two diagonal braces converging at a common point—typically at the beam midspan—forming either an inverted "V" or a conventional "V" shape. In contrast, the X-type bracing system consists of two diagonal members intersecting in an "X" pattern and typically connected via a central gusset plate or directly to the beam. Both Chevron and X-type configurations serve as efficient lateral force-resisting systems and offer redundancy through force distribution among multiple structural elements.

In Buckling-Restrained Braced Frames (BRBFs), the braces incorporate a steel yielding core encased within a steel or concrete restraining mechanism to prevent global or local buckling. Under seismic excitation, BRBs are designed to yield in both tension and compression, thereby facilitating controlled energy dissipation and ductile behavior. The encasement plays a critical role in confining the core and ensuring stability throughout cyclic deformations. A key design consideration in Chevron and X-type BRBFs is the vertical component of force transferred to the beam at the point where the braces intersect. These unbalanced vertical forces—resulting from differential brace actions—can lead to local yielding, deformation, or crippling if not properly mitigated through capacity-based detailing.

The structural elevation illustrated in Figure 1 represents a typical multi-story BRBF system utilizing proprietary CoreBrace buckling-restrained braces (BRBs). The frame demonstrates a performance-based seismic design suitable for high seismic zones, with an emphasis on energy dissipation, system ductility, and predictable inelastic response. The figure illustrates three bracing configurations applied at different levels: inverted V-type Chevron at the roof, V-type Chevron at the second level, and X-type at the first level. Each layout presents unique force paths, connection demands, and design detailing challenges.

At the roof level, an inverted V-type Chevron bracing connection is utilized. In this arrangement, the BRBs are inclined upward from the column bases and converge at a gusset plate positioned near the beam midspan. During lateral loading, the inclined configuration introduces an uplift force at the point of intersection, which must be resisted by the beam-to-column connections, diaphragm anchorage, and the floor slab. This layout is advantageous in scenarios where vertical compression on the beam is undesirable. However, it necessitates careful detailing of the vertical load path, restraint of uplift forces, and appropriate consideration of brace length and eccentricities to maintain seismic performance.

The second level adopts a V-type Chevron bracing configuration, where the BRBs slope downward from the columns to converge at the beam's midspan. This layout is among the most commonly used due to its efficient geometry and high lateral stiffness. Nevertheless, it generates a downward vertical force—referred to as the unbalanced vertical force—on the beam at the brace intersection point. If not properly accounted for, this force may cause web crippling, local flange yielding, or flexural deformation. To mitigate these effects, structural design must ensure that the beam and gusset connections remain elastic through the inclusion of web stiffeners, thicker web plates, or flange reinforcement in accordance with AISC 341-16 and AISC 360-16 provisions.

At the first level, the frame features an X-type bracing configuration. In this system, the BRBs intersect at the center of the bay, forming a symmetric X pattern. This configuration offers balanced lateral resistance in both directions and promotes even force distribution between the supporting columns. It also minimizes vertical load transfer to the beam at the intersection point, which is a key advantage over Chevron systems. However, this geometry introduces connection complexity at the intersection, where proper detailing is required to avoid member interference and ensure sufficient clearance during large inelastic deformations. The use of additional gusset plates, intermediate connectors, or stiffened regions may be necessary to accommodate brace crossover and ensure that the connection meets cyclic performance criteria.

2. Analysis and Design Methods for Chevron Connections in BRBF

Two common analytical approaches for evaluating gusset plate forces in Buckling-Restrained Braced Frame (BRBF) connections are the Uniform Force Method (UFM) and the KISS Method. These techniques are typically employed during different stages of the design process to estimate internal force distribution, size connection elements, and guide detailing of gusset plates.

The Uniform Force Method (UFM), as defined in the *AISC Steel Manual* and AISC 360-16, is a widely accepted code-based approach for the elastic analysis of gusset connections. This method resolves the brace axial force into its horizontal and vertical components, which are assumed to act uniformly on the gusset plate's interfaces with the beam and column. UFM is particularly useful for determining bolt forces, weld requirements, and gusset plate thickness, making it suitable for final design documentation. However, it is based on linear-elastic assumptions and thus does not capture nonlinear behavior such as plastic deformation, buckling, or stress redistribution in the gusset region. Additionally, it cannot identify localized limit states such as yielding at the gusset tip or out-of-plane deformations, which are critical under seismic loads.

The KISS Method—an acronym for *"Keep It Simple, Stupid"*—is a heuristic, non-code approach used primarily during conceptual or preliminary design stages. It simplifies the gusset design process by assuming a 60-degree load spread from the brace into the gusset, creating a triangular region of stress distribution. This method enables us to quickly estimate gusset plate dimensions without detailed calculations, making it attractive for early-stage sizing and layout. However, the KISS Method is not formally recognized by AISC, and it does not provide sufficient information for bolt sizing, weld design, or complex interaction effects. Its use should therefore be limited to preliminary design, and must be supplemented by more rigorous analytical or numerical verification methods.

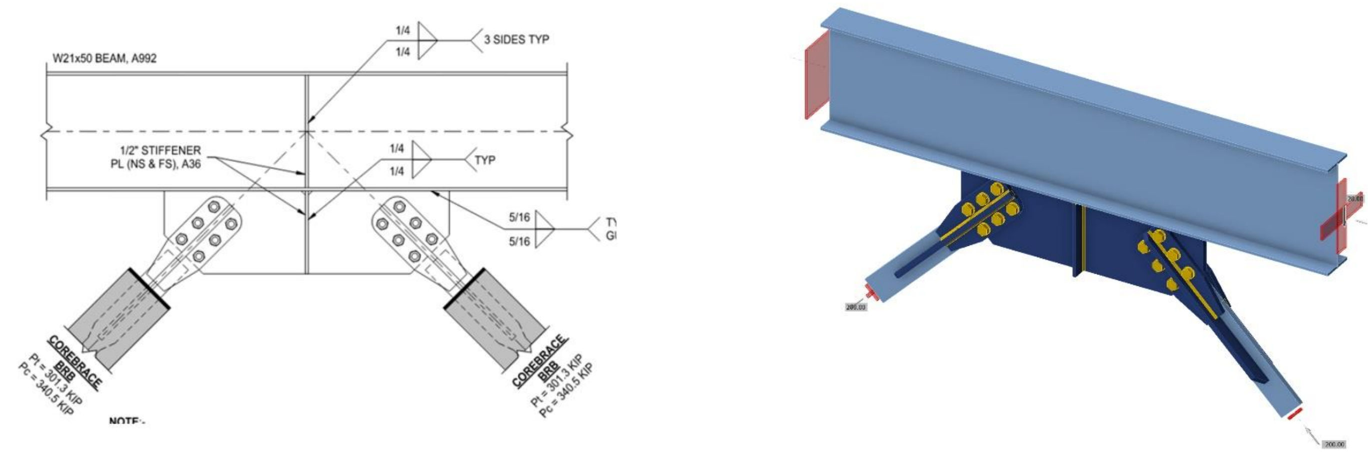
For detailed design and performance validation, especially under seismic loading conditions, engineers increasingly rely on finite element-based modeling approaches, such as the Component-Based Finite Element Method (CBFEM) implemented in software like *IDEA StatiCa Connection*. CBFEM offers a more realistic representation of connection behavior, capturing inelastic deformation, local buckling, bearing failure, and weld/bolt force flow under cyclic loads. It also provides visual feedback on stress distribution and limit state exceedance, which UFM and KISS cannot offer.

A summary comparison of the UFM, KISS, and CBFEM approaches is presented in Table 1, highlighting their key assumptions, applications, limitations, and suitability for different design stages.

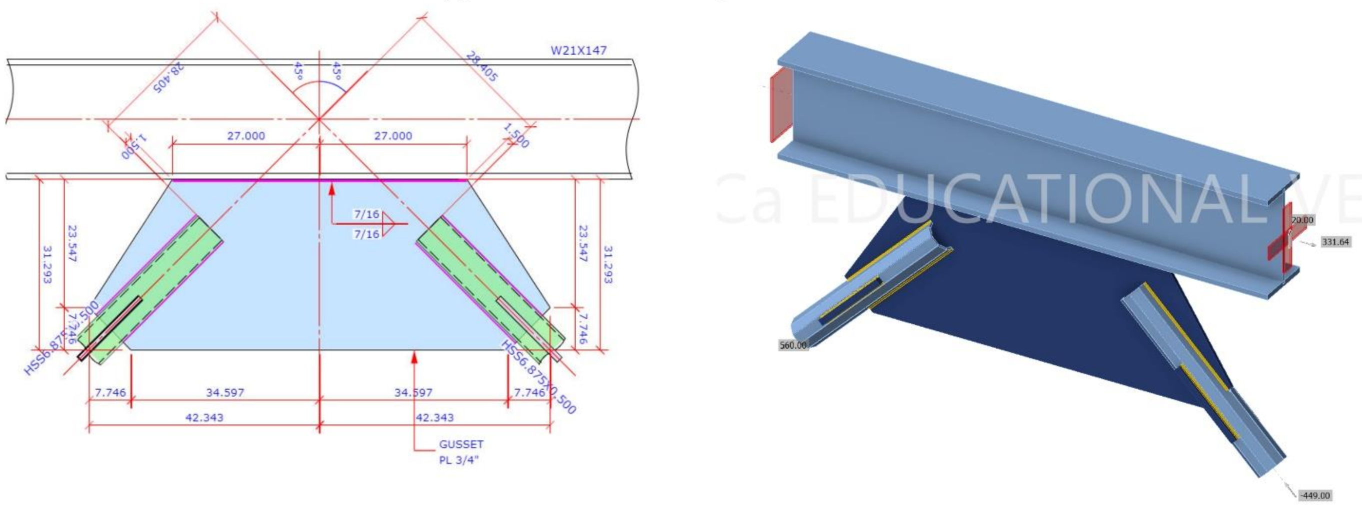
Table 1 – Comparison of UFM , KISS and CBFEM method

Criteria for Comparison	**Uniform Force Method (UFM)**	**KISS Method (Kiss it Simple Stupid)**	**CBFEM Method (Component-Based Finite Element Method)**
1. Purpose of Method	Simplified Analytical Force Calculation for Gusset interface design.	Streamlined Gusset Plate design using fixed angle load paths and sizing rules of thumb.	Non-linear Finite Element Based Connection Analysis in IDEA StatiCa Connection.
2. Analysis Type	Linear Elastic (based on simple rules of statics).	Approximate (based on thumb rules).	Nonlinear Stress/ strain (based on plasticity of elements)
3. Code Alignment	AISC 360-16 / 341-16.	AISC- based on heuristics.	Synergy of Component Method and Finite Element Analysis, Aligns with AISC for Strain and force-based checks
4. Gusset Plate Force Resolution	Uniform axial and shear forces at each interface.	Axial force resolved into orthogonal components at 30-to-60-degree angle.	Real stress flow from defined geometry and stiffness.
5. Plate Deformation Modeled	Not Considered.	Not Considered.	Explicitly modeled and visualized (strain maps).
6. Plastic Strain Checks	No.	No.	Yes, with strain and equivalent strain contours.
7. Weld / bolt force Distribution	Assumed to be Uniform.	Conservative Estimate.	Actual force in each weld/ bolt, considering the defined geometry.
8. Stiffeners, Flange Web Checks	Must be checked individually.	Not Addressed.	Stress and strain in stiffeners, flanges and web included.
9. Gusset Buckling Checks	Requires manual calculations.	Ignored.	First 3 eigen modes computed and displayed.
10. Local Load Path Eccentricity	Ignored.	Ignored.	Modelled based on actual geometry.
11. Typical Use case	Preliminary design, hand checks.	Conceptual gusset plate design.	Final detailed design verification for seismic applications.
12. Result type	Hand-calculated forces, design sizes.	Sizing of gusset plates and bolts.	Utilization ratios, strain maps, Buckling shapes.
13. Output Format	Force tables, equations.	Design tables.	3D model, detailed report with images and utilization factors.

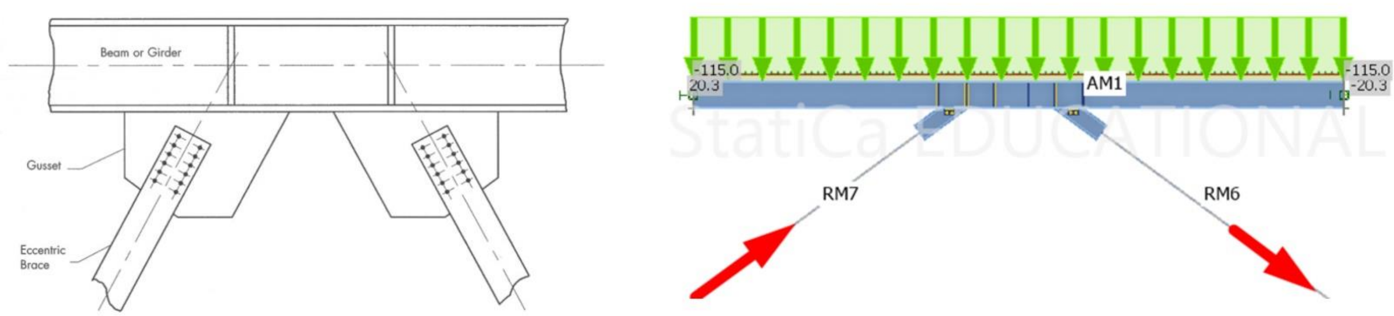
3. Comparison of Chevron Bracing Connections in BRBF, SCBF and EBF



a) Typical Chevron Bracing Connection in BRBF



b) Typical Chevron Bracing Connection in SCBF



c) Typical Link Beam Bracing Connection in EBF

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

Chevron bracing configurations are frequently employed in seismic-force-resisting systems, including Special Concentrically Braced Frames (SCBFs), Eccentrically Braced Frames (EBFs), and Buckling-Restrained Braced Frames (BRBFs) utilizing Chevron-configured Buckling-Restrained Braces (BRBs). Although the geometry of these configurations may appear similar, the seismic behavior, connection detailing, and energy dissipation mechanisms vary significantly due to differing design philosophies outlined in AISC 341-16 and AISC 341-22.

In SCBF systems as shown in Figure 2.b) , Chevron braces are slender elements that yield in tension and buckle in compression during seismic events. This asymmetric brace behavior produces high unbalanced vertical forces acting downward on the beam at the brace intersection. These forces can result in local flange bending, web crippling, and excessive deformation in the beam, which must be addressed through the use of web stiffeners and conservative beam sizing. AISC provisions for SCBFs enforce strict limits on brace slenderness, compactness, and connection detailing to ensure that inelastic behavior remains confined to the braces and that overall system ductility is preserved.

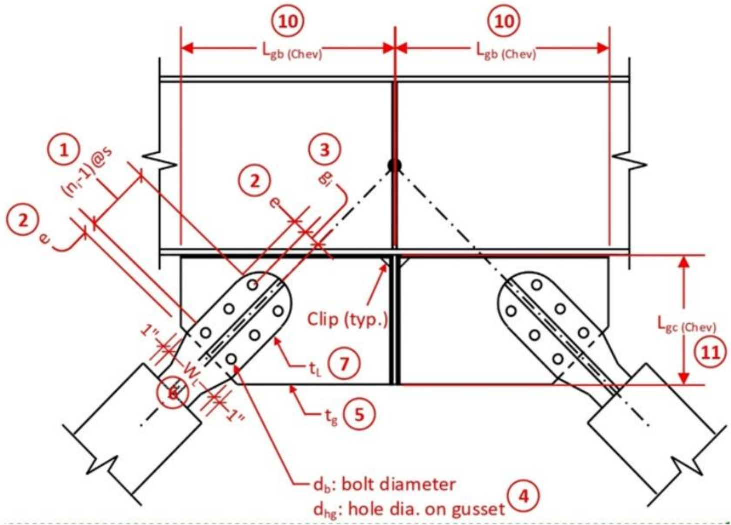
In contrast, BRBF systems as shown in Figure 2.a) incorporate BRBs that yield symmetrically in both tension and compression. This is achieved through the use of a steel core encased in a restraining mechanism that prevents global buckling, resulting in stable hysteretic behavior under cyclic loading. While BRBFs significantly enhance ductility and reduce residual drift compared to SCBFs, they do not eliminate the vertical force transfer to the beam. Since both braces yield concurrently, the beam must resist the sum of their vertical components. This necessitates robust gusset-to-beam connection detailing and careful capacity-based design of surrounding members to ensure elastic performance outside the BRBs.

In EBF systems as shown in Figure 2.c), Chevron braces are connected eccentrically to a short beam segment known as the "link," which is deliberately designed to yield in shear or flexure during seismic events. The braces themselves remain elastic, with inelastic demands intentionally concentrated in the link, offering high ductility and effective damage control. Chevron bracing in EBFs must be proportioned to preserve link action, with design focus on maintaining appropriate eccentricity, limiting axial force transfer into the link, and ensuring sufficient plastic rotation capacity. Detailing efforts also prioritize avoiding local failure mechanisms, such as flange local buckling or web crippling, in the link region.

Unlike EBFs, BRBF Chevron systems do not rely on secondary yielding mechanisms such as link deformation. Instead, the BRBs themselves act as the primary energy-dissipating elements. This simplifies the structural force path but imposes higher axial and vertical demands on the gusset plate and beam-column interfaces. Additionally, BRBF systems require rigorous BRB qualification testing to validate performance under expected loading scenarios. Although Chevron bracing appears geometrically consistent across SCBF, EBF, and BRBF systems, each approach requires a tailored connection strategy, force transfer mechanism, and performance verification method aligned with the intended ductile behavior of the system.

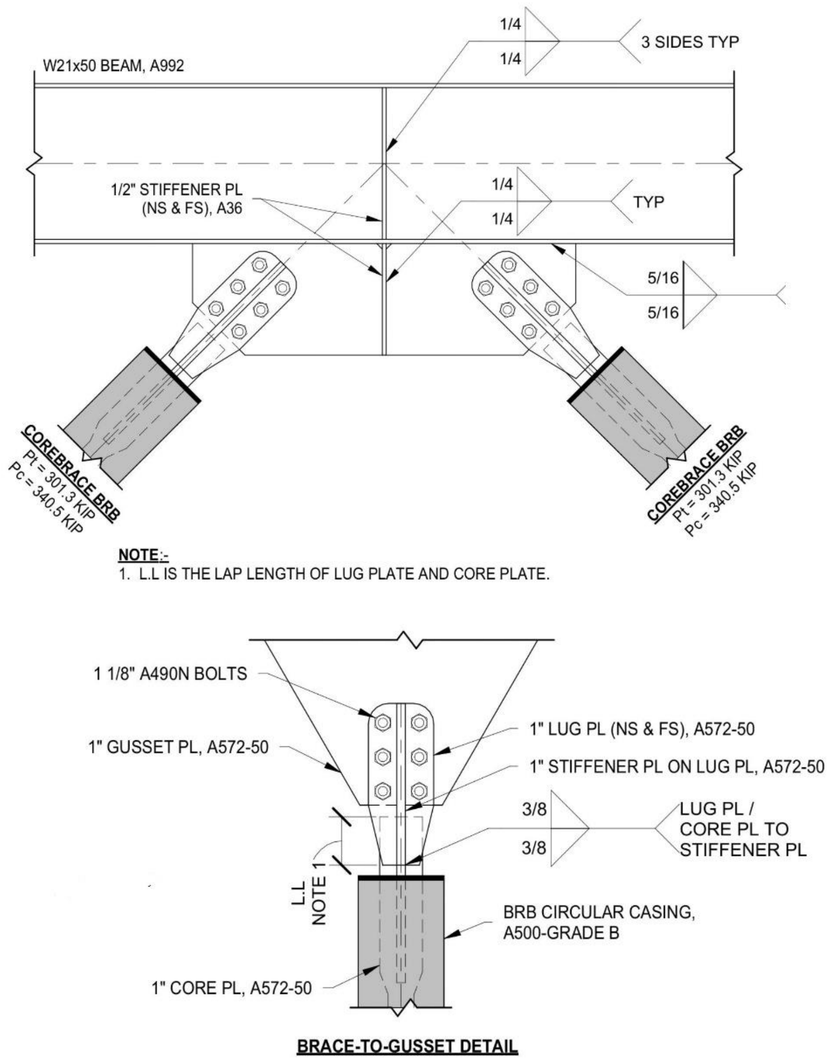
4. Problem Description

The objective of this example is to verify the Component-Based Finite Element Method (CBFEM) for a Chevron bolted-type single-cored bracing 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.

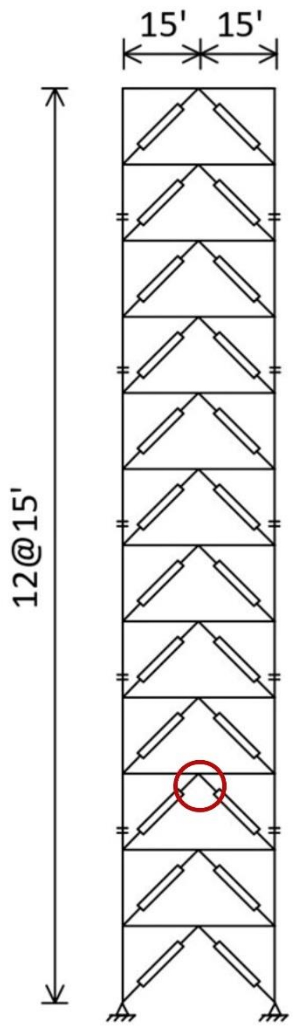


a) General Sketch for Chevron Type BRB Connection

b) Image of Constructed Chevron BRB Connection



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



d) Building Elevation and Location of Considered Connection

Figure 3 – Bolted-Type Chevron CoreBrace Connection Details

Figure 3 illustrates the configuration, fabrication, and design detailing of a Chevron Buckling-Restrained Brace (BRB) connection using a bolted lug-type assembly, as applied to a real building frame. Here's a detailed explanation of each subfigure (a) through (d):

a) General Sketch for Chevron Type BRB Connection

This schematic shows a plan view of a typical Chevron BRB configuration using two inclined BRBs meeting at a gusset plate and connecting to a horizontal beam. Important features include:

- Lug Plates: The end fittings of the BRBs which connect via high-strength bolts to the gusset plate.
- Bolt Layout : The sketch notes the bolt diameter and hole diameter, critical for calculating bolt bearing and tear-out strength.
- Lap Length (LL): The length of overlap between the lug plate and core plate ensures full force transfer from the brace core to the connection.

b) Image of Constructed Chevron BRB Connection

This real-world photograph shows the installed Chevron BRBs in a building. Key observations:

- The beam is stiffened at the connection zone, likely due to unbalanced vertical forces common in Chevron configurations.
- Gusset plates are bolted to the brace lugs using symmetrical bolt arrangements.
- Field bolting is evident, highlighting the practicality and modularity of bolted lug BRB systems for construction.

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

This subfigure presents a detailed design drawing compliant with AISC provisions.

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 (F_{ysc}) range of core plate = 38 ksi to 46 ksi
- 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 End Connection for Brace-to-Gusset.
2. Welds (Fillet welds at all applicable locations)
- E70xx electrode
- 3/8” double sided fillet weld between stiffener plate and lug plate.
- 5/16” double sided fillet weld between gusset plate and bottom 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 = 12.38 in.
- a = Gap between core and gusset = 4 in.

2. **Stiffener Plates in beam and gusset**

- PL ½", ASTM A572-50

3. **BRB Core Plate**

- PL 1", ASTM A572-50

4. **Lug Plate**

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

3. Bolts

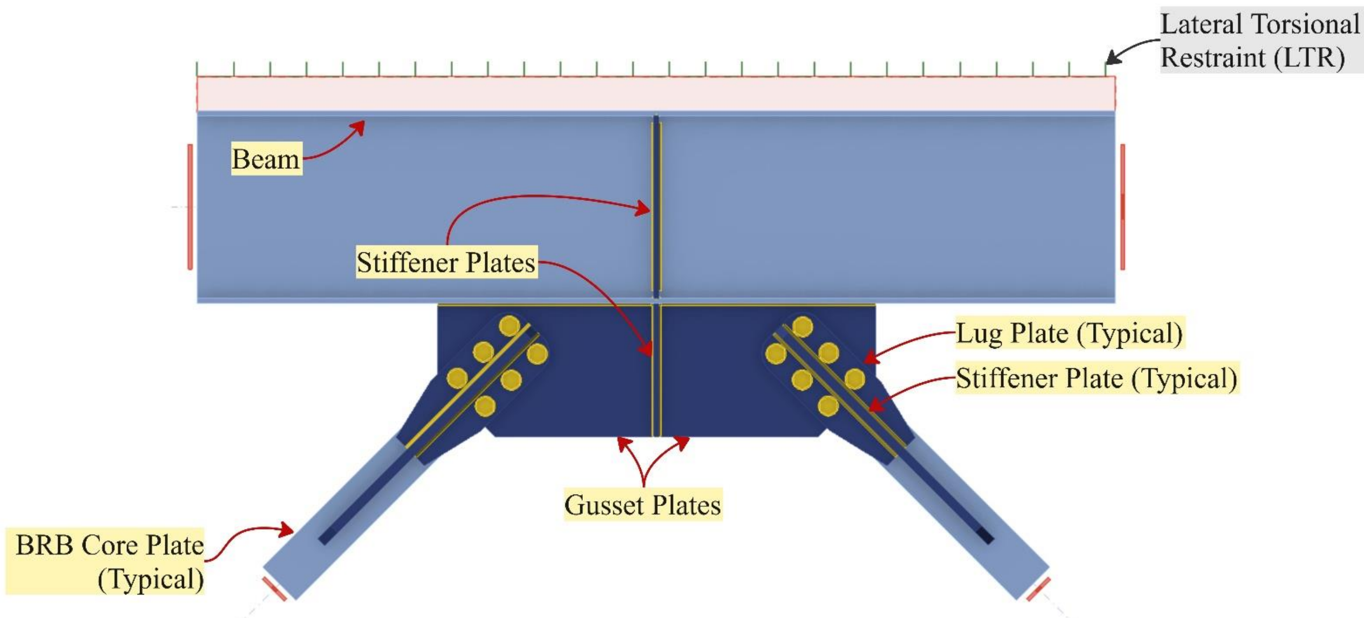
- 1 1/8" A490-N type bolts connecting lug plate to gusset plates.
- n_i = Number of bolts in inner row = 3
- n_o = Number of bolts in outer row = 0
- g = Gauge between outer and inner bolt rows = 2 3/16 in.
- s = Bolt spacing = 4 in.
- e = Typical bolt edge distance = 1.63 in.
- b_r = Distance to start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

d) Building Elevation and Location of Considered Connection

This frame elevation drawing shows a 12-story BRBF structure with Chevron bracing in each bay. The red circled area identifies the specific connection being studied—located on the 6th level of a Chevron BRBF bay.

5. Modelling and Analysis of Connection in IDEA StatiCa

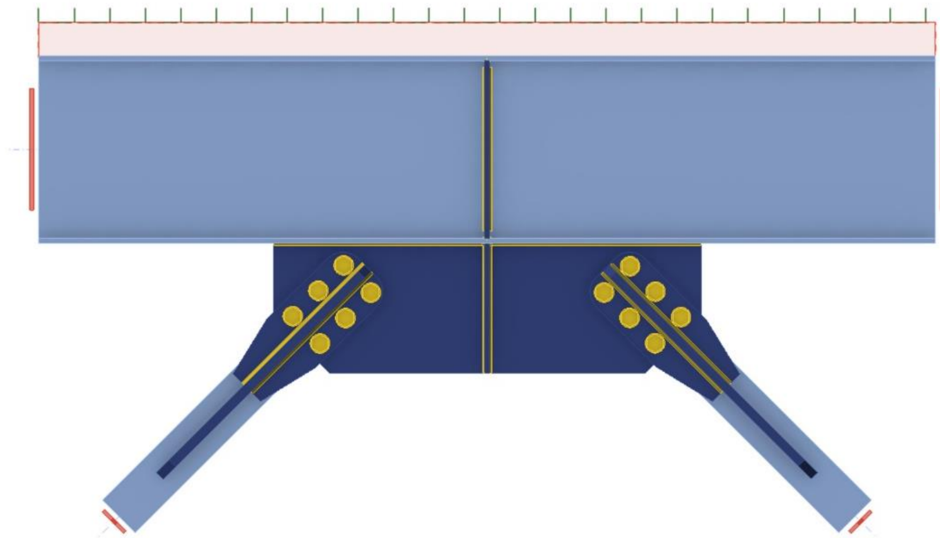
The connection modeling in IDEA StatiCa begins with defining the members—specifically, the beam and two BRB core plates. Next, the two gusset plates and two stiffener plates are defined and arranged according to the connection details shown in Figure 2. The top of beam is assigned lateral torsional restraint (LTR), to account for composite deck. The BRB 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.



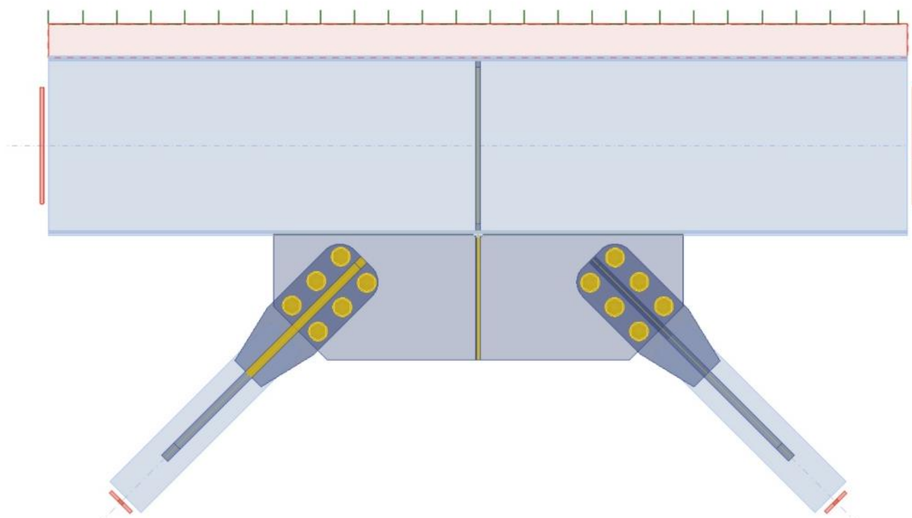
a) Defination of elements in base model for bolted CoreBrace BRB chevron connection – CBFEM

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

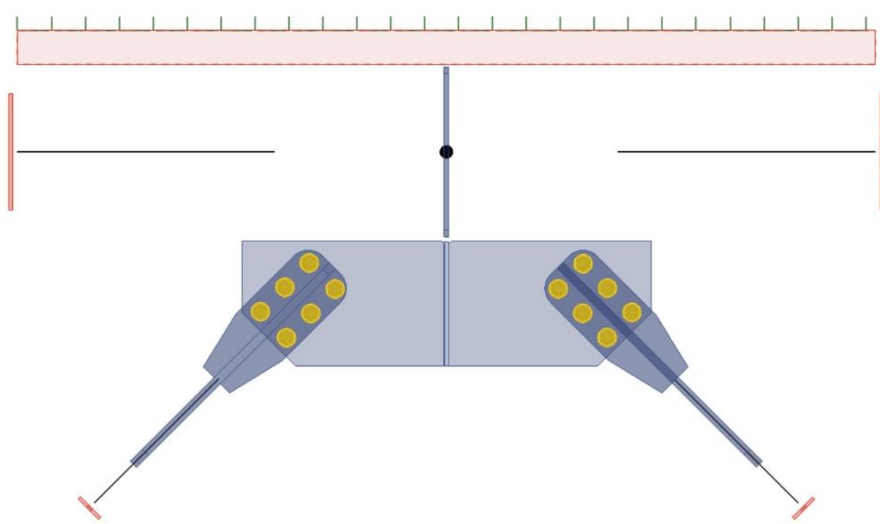
The beam is assigned as a bearing member in the CBFEM model. A model type of N-Vy-Vz-Mx-My-Mz (Fixed) is assigned to the beam, while the two bracing members are assigned a model type of N-Vy-Vz (Pinned). This setup ensures that forces are applied at the nodes, as shown in the wireframe view in Figure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



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



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

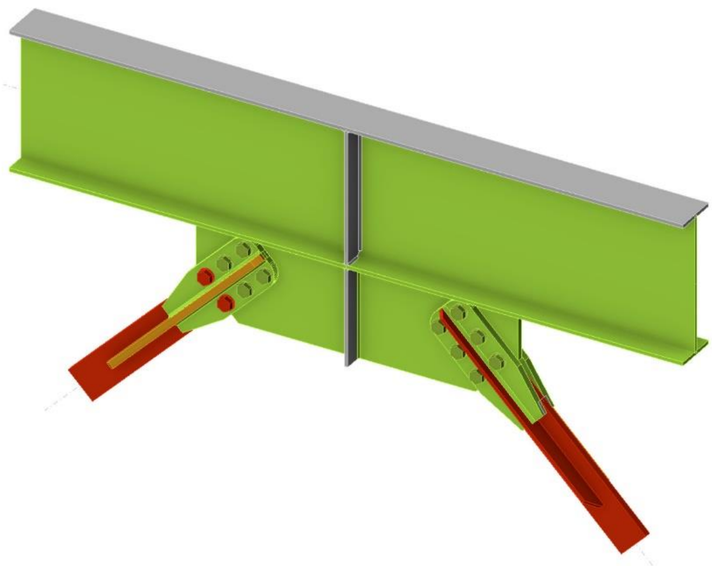


c) Wireframe view of CoreBrace BRB Chevron base model - CBFEM

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 5, the conclusions are summarized in Table 2 below.

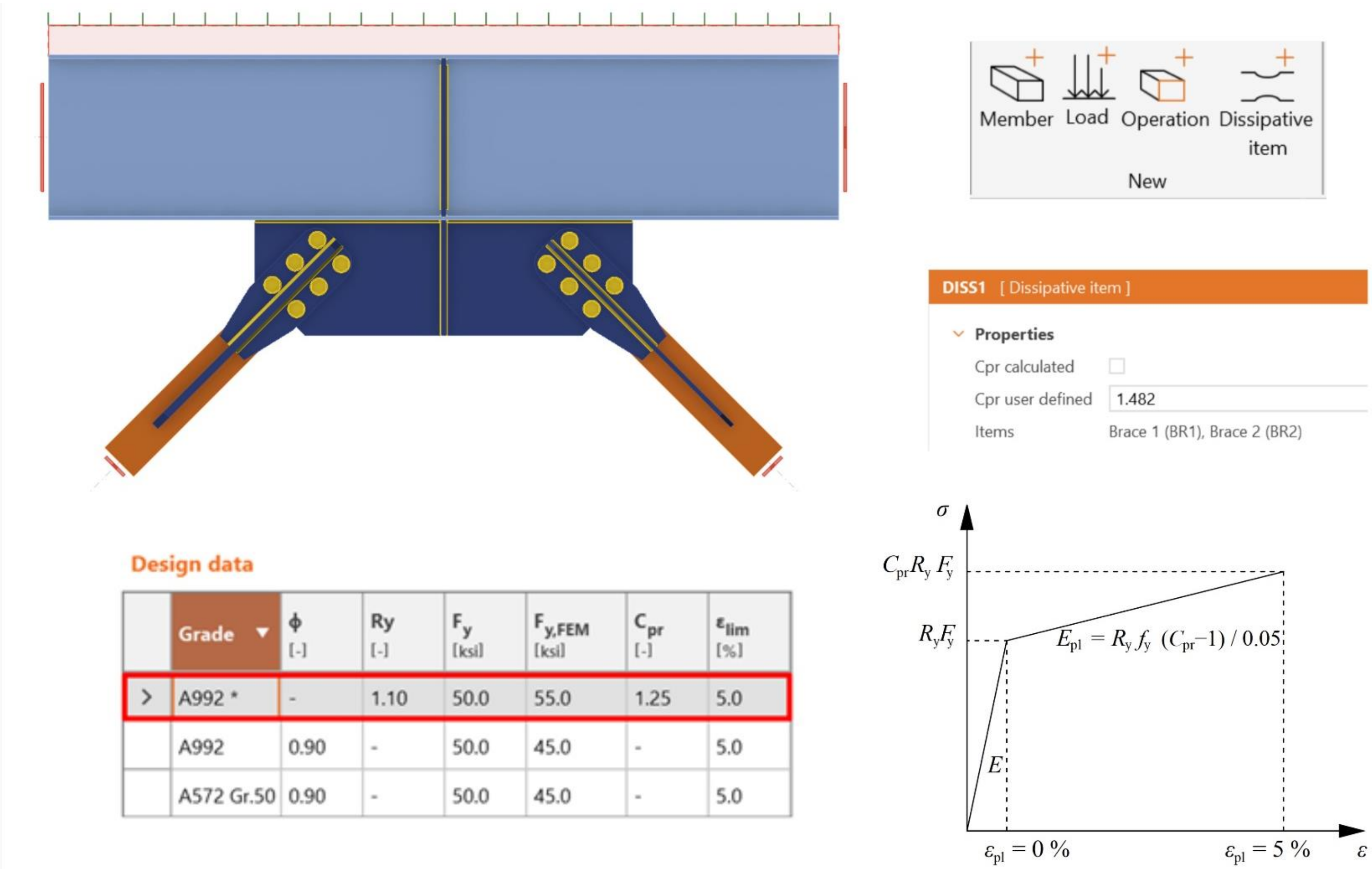
Table 2 – Traffic Light Visualization of the Chevron BRB Connection in CBFEM

Sr No	Color	Elements	Comments
1	**GREY**	BRB Core member	Elements are safe for action of loads.
2	**GREEN**	Stiffeners in beam, Bottom flange of beam, Knife plates on brace etc.	Elements are safe for action of loads.
3	**ORANGE**	Top flange of beam, Web of beam etc.	Elements are on the verge of failure, for the applied loads.
4	**RED**	Gusset plates, bolted connection etc.	Elements are failing for the applied loads; utilization is beyond 100%.

Capacity design analysis is performed for BRB connection in CBFEM. Both the core brace members are 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 and Compression Load

A capacity design analysis was performed for the Chevron BRBF connection using the Component-Based Finite Element Method (CBFEM) implemented in IDEA StatiCa Connection (Version 25). In accordance with the BRBF design philosophy outlined in AISC 341-16, brace members were defined as dissipative elements, intended to yield under seismic loading, while the surrounding components—such as gusset plates, beam interfaces, welds, and stiffeners—were treated as non-dissipative, expected to remain elastic or experience limited inelasticity. The objective of this verification was to evaluate the strength and deformation capacity of the connection components under progressive axial loading applied through the BRBs.

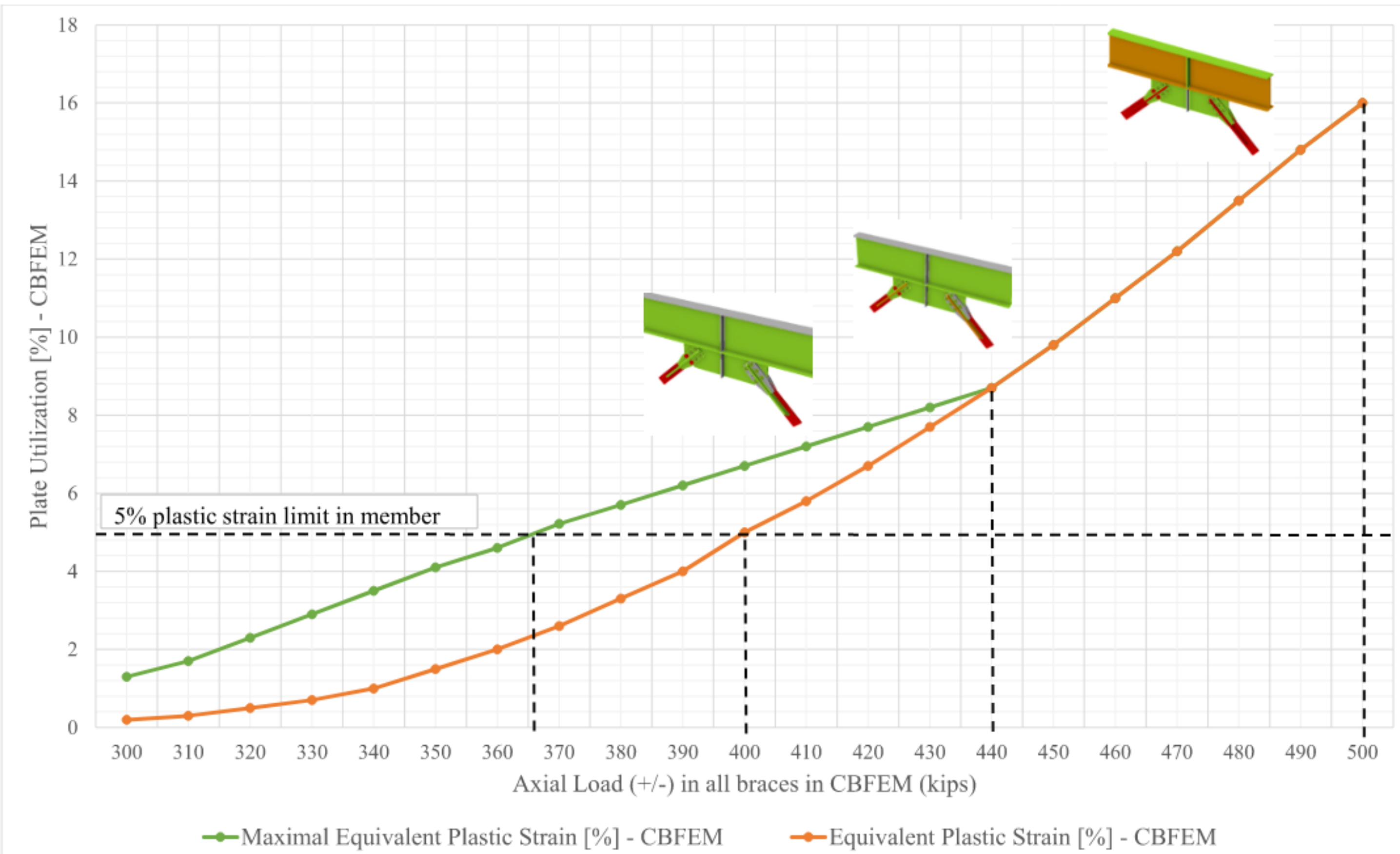


Figure 8 - Graph for Evaluation of Capacity of Elements – CBFEM

To assess this, a parametric axial loading procedure was performed in which axial forces ranging from 300 kips to 500 kips were incrementally applied to braces. The analysis continued until critical connection elements exhibited equivalent plastic strains at or above 5%, a limit commonly used in CBFEM. For interpretive clarity, the strain behavior within the most critical loading range—300 to 500 kips—is presented in Figure 8, which employs a traffic light–style visualization format to indicate performance thresholds.

Two primary strain metrics were monitored:

- Equivalent plastic strain ($\varepsilon_{pl,avg}$), representing the average plastic deformation across yielding zones, and

- Maximum equivalent plastic strain ($\varepsilon_{pl,max}$), identifying the peak strain in the most critically overstressed element.

As shown in Figure 7, the connection remained within the elastic regime up to approximately 365 kips, beyond which the $\varepsilon_{pl,max}$ curve crossed the 5% limit, indicating initial yielding in the brace member—a favorable and intended outcome in BRBF systems. As axial load increased further to 440 kips, the shop-welded stiffener plate attached to the lug plate began to exhibit localized failure, with average plastic strain reaching 5.5% and peak strain rising to 6.5%, highlighting overstress in this connection zone. At the upper bound of the applied loading—500 kips—the simulation revealed significant plastic strains in both the web and bottom flange of the beam, with local strains escalating to approximately 16%, suggesting that the connection had entered a post-ductile failure regime, likely governed by rupture or instability in the beam component.

In practice, it is entirely acceptable—and often expected—for the maximum equivalent plastic strain and the average equivalent plastic strain to differ in a CBFEM analysis, particularly within the context of capacity design, where inelastic deformation is intentionally concentrated in the dissipative element (e.g., the brace for BRBF). This divergence arises due to factors such as mesh refinement, geometric discontinuities, and stress concentrations in localized features like weld toes, bolt holes, or stiffener transitions. The critical consideration lies not in the numerical difference itself, but in its interpretation. If the maximum plastic strain is relatively high but isolated within a small, non-structural region—such as a weld toe in a stiffener plate—and the average plastic strain remains below the prescribed threshold (e.g., 5%), the connection is generally considered structurally sound. On the other hand, if high strain values are distributed across multiple non-dissipative components, it indicates a loss of confinement of inelasticity, suggesting that the connection may be vulnerable to premature or brittle failure. In such cases, the design must be re-evaluated and possibly strengthened to ensure that yielding remains concentrated in the intended energy-dissipating elements and that global connection integrity is preserved under seismic loading.

The evolution of the strain curves provides critical insight into the redistribution of plastic deformation. Between 370 and 440 kips, the gradual convergence of the average and maximum strain curves indicates that initial localized yielding in specific elements transitioned into a more distributed and uniform inelastic response. Beyond 440 kips, the near-parallel rise of both curves confirms that plasticity had propagated across multiple critical regions, resulting in a more consistent yielding mechanism. These findings are supported by CBFEM-generated visual strain snapshots, which clearly illustrate the progression from initial yielding at gusset interfaces to more widespread plastic deformation across welds, bolt holes, and beam flanges.

7. Verification of Resistance - Capacity of Connection

For the given connection, various elements such as plates and structural members are joined using fillet welds of varying sizes and lengths. Following the evaluation of element capacities, the welded connections were assessed using the same models and details.

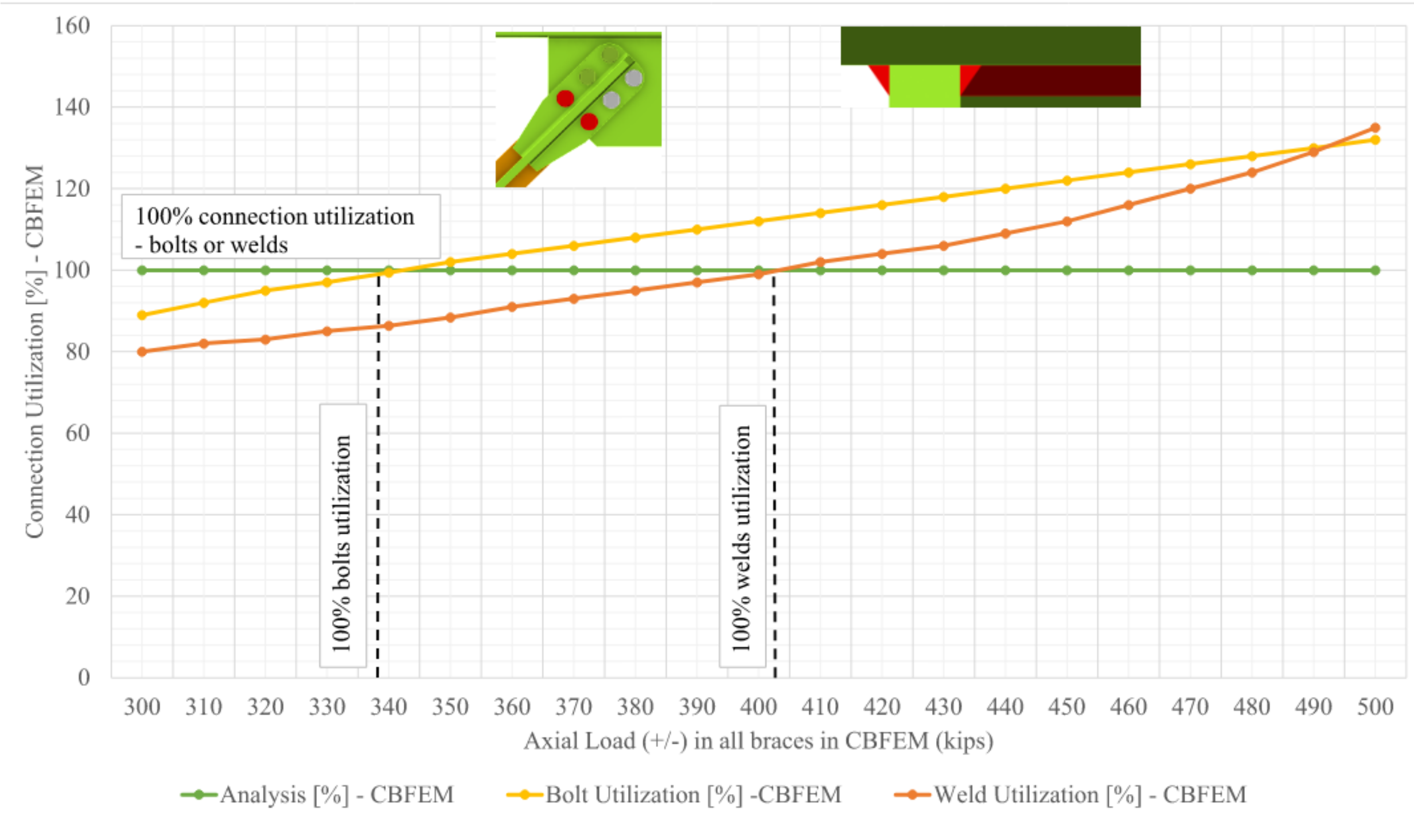


Figure 9 - Graph for Evaluation of Capacity of Connection – CBFEM

Figure 9 presents a connection capacity evaluation based on the utilization of bolts, welds, and overall connection strength as determined through Component-Based Finite Element Method (CBFEM) analysis using *IDEA StatiCa*. The x-axis represents the total axial load in all braces of the Chevron BRBF connection, ranging from 300 kips to 500 kips, while the y-axis shows the connection utilization percentage, where 100% utilization represents the theoretical capacity limit for bolts or welds. Three distinct curves are plotted: the green curve represents the overall connection utilization from CBFEM analysis, the yellow curve indicates bolt utilization, and the orange curve corresponds to weld utilization. Two vertical dashed lines are shown to mark the points at which bolt and weld utilization each reach 100%, denoting their respective limit states.

As observed from the graph, bolt utilization reaches 100% at approximately 340 kips, indicating that the bolted connection has reached its design capacity at this axial load. Beyond this point, any further increase in axial force could potentially lead to bolt failure unless additional bolts, higher-strength fasteners, or slip-critical provisions are provided. Similarly, weld utilization reaches the 100% threshold near 400 kips, suggesting that the welds approach their capacity slightly later than the bolts. Beyond 400 kips, the orange curve continues to rise, exceeding the design limit and indicating that the welds may be vulnerable to overstress under higher brace forces.

Interestingly, the overall analysis curve (green) remains steady at approximately 100% after 340 kips, implying that while individual components (bolts and welds) are becoming overstressed, the global connection behavior as modeled in CBFEM remains stable within acceptable thresholds—up to a point.

This divergence between global connection behavior and individual component utilization is crucial in capacity design. It highlights that local failures may govern the connection's performance long before the entire assembly reaches collapse. In this case, both bolt rupture and weld yielding are critical limit states that must be addressed through proper detailing, such as increasing weld throat size or upgrading bolt class. The embedded model snapshots in the graph further visualize regions of overstress, showing red strain zones forming in the gusset-to-lug interface (bolt locations) and weld seams as axial loads approach and exceed the 400–500 kip range.

8. Code check of connection in CBFEM

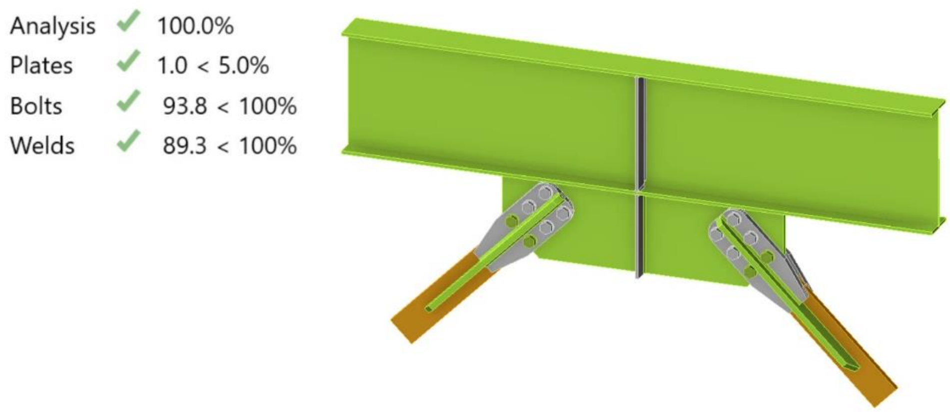
Proportioning the braces to their adjusted strength is a recommended practice by AISC 341-16 for Buckling-Restrained Braced Frames (BRBFs). The adjusted brace strength in tension ($P_{tension}$) shall be $\omega R_y P_{ysc}$, while the adjusted brace strength in compression ($P_{compression}$) is calculated as $\beta\omega R_y P_{ysc}$. The values for adjustment factors (β, ω) and design factor (R_y) are provided by brace manufacturer (CoreBrace).

where,

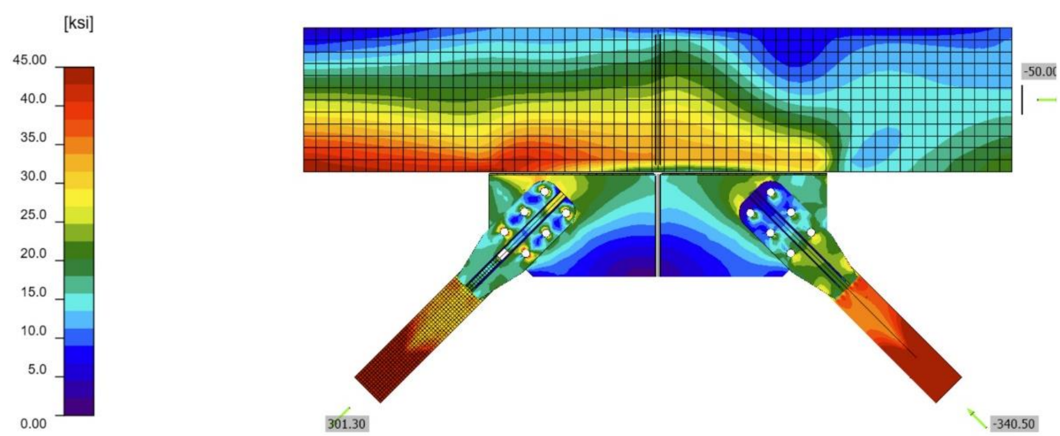
- β is compression strength adjustment factor
- ω is the strain hardening adjustment factor
- R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y
- P_{ysc} is the axial yield strength of steel core, ksi (MPa), calculated as $P_{ysc} = F_{ysc}A_{sc}$, where F_{ysc}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

As stated in Section F4.3, the required strength of columns, beams, struts, and connections in a Buckling-Restrained Braced Frame (BRBF) is determined using capacity-limited seismic load effects. These effects are defined as the forces developed in the members, assuming that all braces are subjected to their adjusted strength in either compression or tension. Braces are to be considered in compression or tension without accounting for gravity loads. The design loads, along with the connection details used in the analysis, are shown in Figure 2.

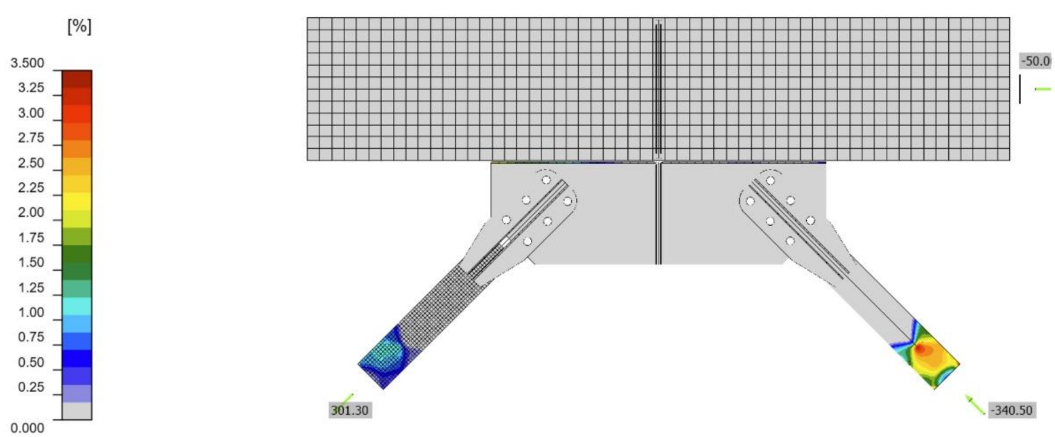
The capacity design analysis of the given connection is performed under the action of design loads in the connecting members using CBFEM. All brace members are designated as dissipative elements in the capacity design.



a) Overall Check of CoreBrace BRB Chevron Connection - CBFEM



b) von Mises stresses of CoreBrace BRB Chevron Connection - CBFEM



c) Plastic Strain of CoreBrace BRB Chevron Connection - CBFEM

Figure 10 – Evaluation of Chevron CoreBrace BRB Connection for Design Loads - CBFEM

Figure 10 presents the comprehensive CBFEM evaluation results for the CoreBrace Buckling-Restrained Brace (BRB) Chevron Connection under seismic capacity design loading. The top panel (a) shows the overall connection check summary, confirming that the connection meets all performance criteria as per AISC 341-16 and IDEA StatiCa's verification framework. The overall analysis utilization is reported at 100%, meaning that the connection has reached the prescribed seismic load level without exceeding any limit states. The utilization of plates is well within permissible strain limits, with a maximum strain of only 1.0%, significantly below the 5% strain threshold typically used for evaluating ductility and local yielding in seismic applications. Bolts are utilized up to 93.8% of their design capacity, and welds are at 89.3%, indicating that both fastening systems are functioning within their allowable stress limits, and no overstress or rupture is anticipated under the given loading.

Subfigure (b) displays the von Mises stress distribution across the connection in ksi, highlighting stress concentrations near the brace-to-gusset interface and around the bolt holes—which is typical and expected in Chevron configurations due to the high axial load transfer. Despite localized peaks, the stress values across the gusset, stiffeners, and beam elements remain within elastic limits (below the yield stress of 46 ksi for the CoreBrace steel core), ensuring that non-dissipative components remain in the elastic range as intended. Subfigure (c) presents the equivalent plastic strain distribution, which confirms that plastic deformation is localized only in the brace end, just outside the gusset-to-lug connection. The maximum observed plastic strain is approximately 3.5%, well below the 5% limit commonly adopted in CBFEM as an upper bound for stable ductile behavior. This confirms that the brace is yielding as intended, and all other components—such as gusset plates, welds, and bolts—are performing elastically, preserving the hierarchy of strength dictated by capacity design principles.

Collectively, the outputs in Figure 10 validate that the Chevron BRBF connection is functioning as intended in a ductile seismic system: yielding is concentrated in the BRB core, while surrounding elements maintain elastic or near-elastic response. The strain and stress levels observed across plates, welds, and bolts confirm that no component is overloaded, and no premature failure mechanisms are present.

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

The calculations based on AISC Specifications were carried out using the provisions for Load and Resistance Factor Design (LRFD) to evaluate the various connection limit states. For the CBFEM method, limit states were investigated individually through a series of iterations by monitoring the distribution of plastic strains and equivalent von Mises stresses. The corresponding connection capacities were then determined using IDEA StatiCa Version 25.0.

The maximum permissible axial loads were identified iteratively by incrementally adjusting the applied load input. This process continued until the model transitioned from a safe to an unsafe state, defined by IDEA StatiCa as exceeding the acceptable stress or strain limits. Load increments of 1 kips were used to pinpoint this transition. It is important to note that this verification study focuses solely on limit states related to the connection.

9.a. Tensile Yielding in Gusset Plates

Tensile yielding is defined as gross section yielding of the gusset plate under the action of the applied axial tension from the brace. In a lug-type gusset plate, tensile yielding occurs along the gross cross-section of the plate perpendicular to the direction of the brace axial force. For this limit state, bolt holes are not deducted from the area, as yielding is assumed to be initiated in the gross cross-section.

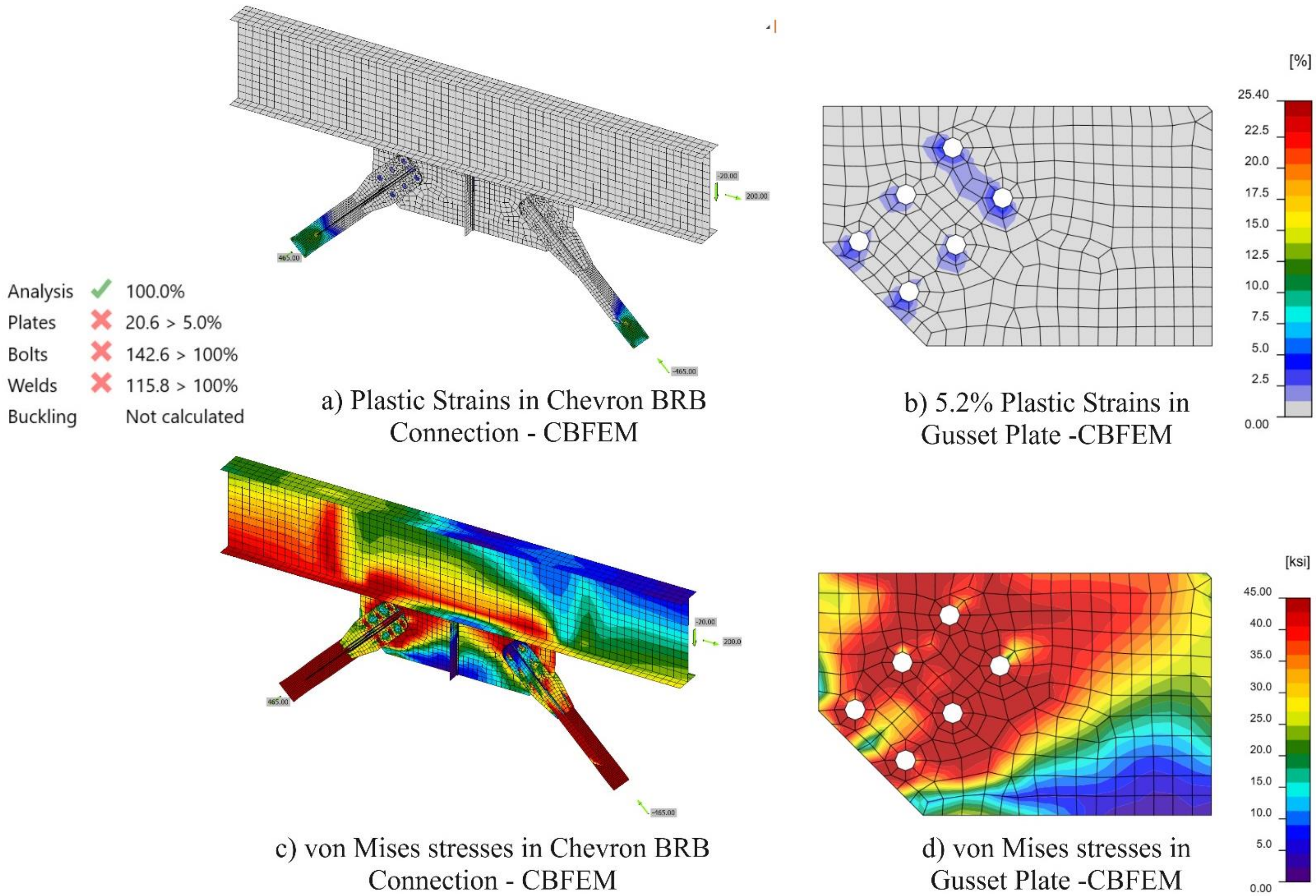
According to Section D2(a) of AISC 360-16, the design strength for tensile yielding is given by:

Tensile Yielding Strength (LRFD):

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

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: $A_g = t \cdot w$
- t is the thickness of gusset plate
- w is width across the gross yielding plane



a) Plastic Strains in Chevron BRB Connection - CBFEM

b) 5.2% Plastic Strains in Gusset Plate -CBFEM

c) von Mises stresses in Chevron BRB Connection - CBFEM

d) von Mises stresses in Gusset Plate -CBFEM

Figure 11 – Evaluation of Limit State of Tensile Yielding in Gusset Plate – CBFEM

The limit state of tensile yielding was clearly captured in the CBFEM analysis performed in IDEA StatiCa at an axial load of approximately 465 kips in each brace, as shown in Figure 11. When compared with the design strength computed using the AISC 360-16 provisions, which yielded a capacity of 450 kips, the CBFEM results are slightly higher—showing a 3% deviation. This difference is within acceptable engineering tolerances and is attributable to the underlying differences in methodology: AISC equations assume uniform stress distribution across the yielding plane, whereas CBFEM accounts for the nonlinear stress redistribution that naturally occurs in thick or stiff plates. Furthermore, CBFEM permits localized plastic deformation (up to 5% plastic strain) before identifying failure, in contrast to AISC, which defines capacity at the onset of first yield.

Although tensile yielding rarely governs the design in Chevron BRBF gusset plates—where limit states such as block shear or bolt rupture are typically more critical—it remains a required check to ensure compliance with capacity design principles. The CBFEM findings confirm that the gusset plate operates safely under this limit state, with no excessive plastic strain observed, thereby validating both the analytical model and the detailing used in the connection. Provided that plastic strain thresholds remain within acceptable bounds (≤5%), the connection is considered structurally safe under this tensile yielding criterion.

9.b. Block Shear Rupture in Gusset Plates

Block shear rupture is a potential failure mode in gusset plates connected to Buckling-Restrained Braced Frame (BRBF) members through bolted lug plates. It occurs when a portion of the gusset plate tears out due to a combination of tension and shear stresses around the bolt group.

To evaluate the block shear rupture limit state for the chevron 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

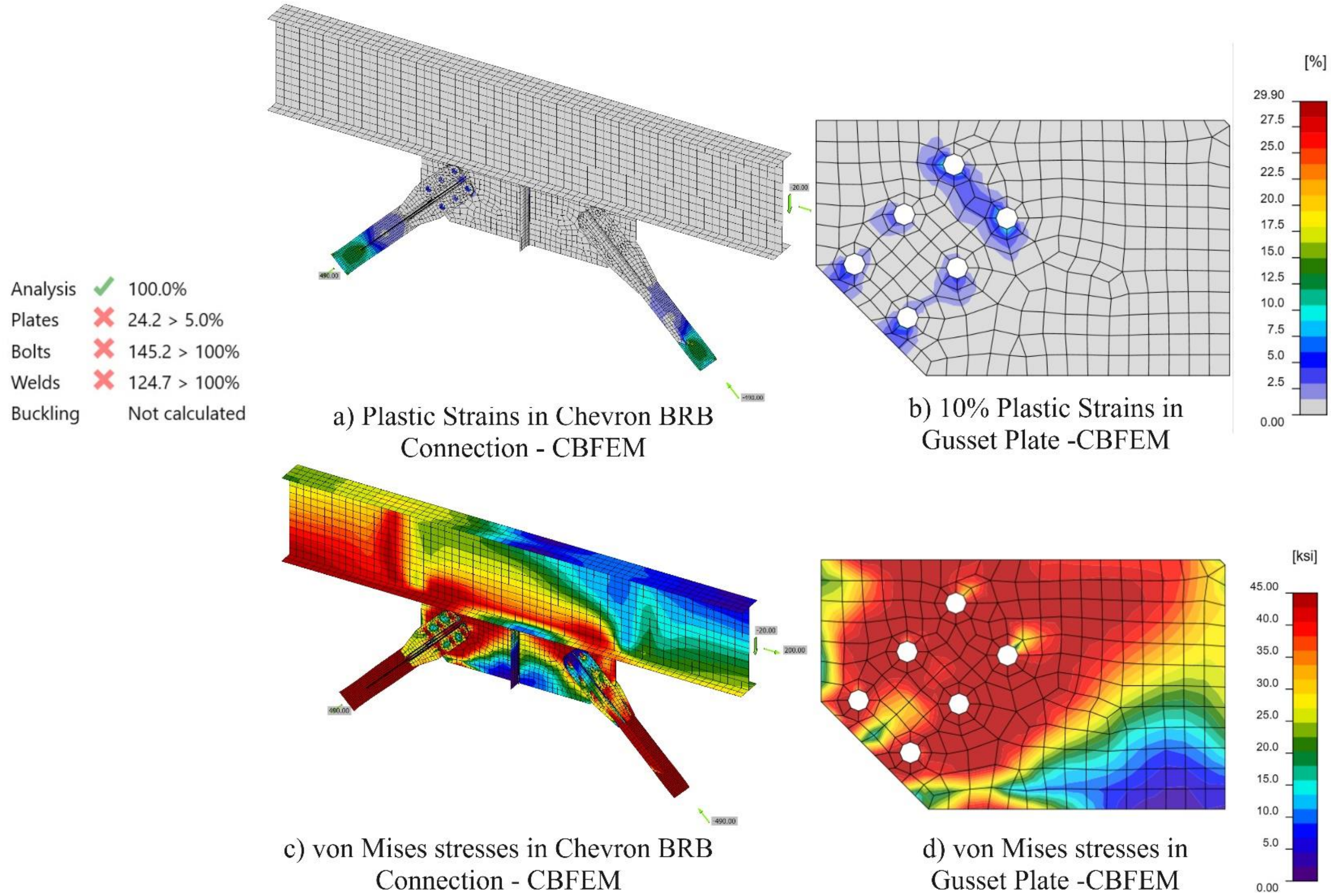


Figure 12 – Evaluation of Limit State of Block Shear in Gusset Plate – CBFEM

In the CBFEM analysis of the Chevron BRBF connection, the gusset plates were subjected to incrementally increasing axial loads applied through the brace members to evaluate the onset and progression of block shear rupture. As illustrated in Figure 12, this limit state was observed in the gusset plate located above the beam at an axial load of approximately 490 kips, which represents a 1% increase over the predicted capacity of 485 kips calculated using the AISC 360-16 equations. The observed failure surface consisted of shear yielding along the bolt lines and tensile rupture across the reduced net section between the last row of bolt holes and the plate edge—consistent with typical block shear failure modes.

The strain concentration contours generated in the CBFEM plastic strain analysis clearly demonstrated the interaction between shear and tensile yielding along this failure path. Unlike the conventional AISC method, which treats the shear and tensile planes as separate and combines their effects conservatively using linear elastic assumptions, CBFEM captures the full interaction between stress fields in both directions, allowing for a more realistic representation of failure behavior. Specifically, CBFEM does not restrict stress to a single critical section, but rather distributes it across the gusset plate's actual geometry—considering factors such as bolt group stiffness, plate thickness, and bolt edge distances, all of which affect the initiation and spread of rupture.

A key distinction lies in the treatment of material nonlinearity: AISC methods typically assume failure occurs at the first yield point, while CBFEM permits limited plastic strain development (up to 5%) before classifying a component as failed. This enables CBFEM to predict slightly higher load capacities, as some post-yield behavior is captured—especially beneficial for gusset plates with compact geometries or redundant load paths. Moreover, by capturing the non-uniform and progressive nature of block shear rupture, CBFEM more efficiently utilizes the available material strength and better represents actual behavior under seismic loading.

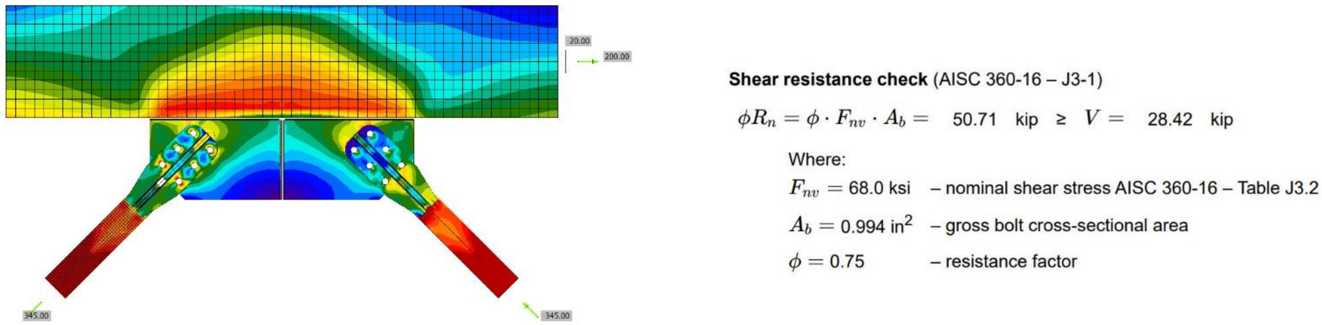
Overall, the close agreement between the CBFEM prediction and the AISC-based calculation provides strong validation of the model and reinforces the reliability of both methods for evaluating this limit state. Although block shear rupture may not always govern the design, it remains a critical check—particularly in bolted gusset-lug connections, where tight bolt spacing, limited edge distances, and shear path constraints can increase vulnerability to this failure mode.

9.c. Bolt Shear

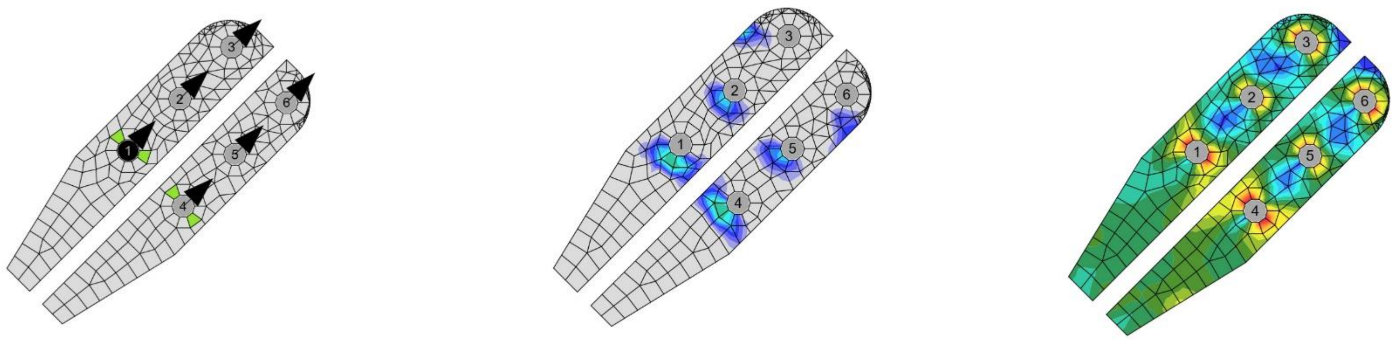
The bolt shear limit state refers to the failure of a bolt due to excessive shear force acting across its cross-section. It must be checked whenever bolts are used to transmit shear forces in a connection (e.g., gusset-to-brace, lug-to-gusset, or gusset-to-beam connections in BRBF systems). For the presented connection, the limit state of bolt shear will be evaluated for the bolts connecting the lug plate to the gusset plate.

As per AISC 341-16, bolts are required to be pretensioned, and they must be designed as slip-critical if used in a Seismic Force-Resisting System (SFRS). Combined tension and shear interaction is checked using standards like AISC Equation J3-6.

Bolts in CBFEM are modeled as nonlinear springs connecting plates. Each bolt has stiffness in shear and tension, and its behavior under load is tracked individually. Bolt geometry, location, and pretension (if defined) are all taken into account. Bolt shear is calculated for one bolt — both in theory and in software — and then compared to its capacity. For groups of bolts, we must check all bolts individually, as one may govern the limit state.



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

Figure 13 – 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 brace force is resolved into components, with the horizontal component often being the major contributor to bolt shear. The lug plate transmits this force into the gusset via shear in the bolts as shown in Figure 13.

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:

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

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

$$\boldsymbol{R_n = F_{nv} \cdot A_b}$$

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):

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

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

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

The value of bolt shear obtained using the AISC equation (50.6 kips) is similar to the value obtained from CBFEM (50.7kips) for the most highly stressed bolts, where 100% bolt utilization is reached under an axial tension load of 345 kips, as shown in Figure 13.

9.d. Bolt Bearing

In Buckling-Restrained Braced Frame (BRBF) systems using bolted lug connections, bolt bearing is another critical limit state to check—especially where brace axial loads are transferred from the lug to the gusset through bolts. Bearing failure occurs when the plate material surrounding the bolt hole deforms excessively or tears due to localized compression.

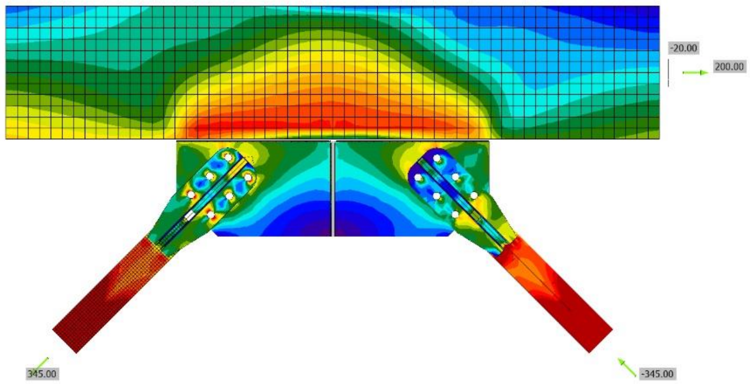
When bolts connect two plates (e.g., lug to gusset), each plate can fail in bearing independently, depending on plate thickness, material strength, and edge distance. Since the bearing capacity is a function of each plate's properties, both sides of the bolt group must be checked separately. Refer to AISC 360-16, Section J3.6 – Bolt Bearing Strength for the applicable design provisions.

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

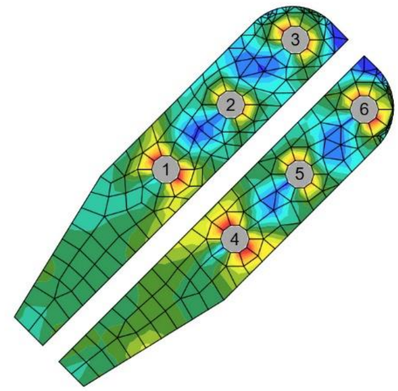
Where:

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

In IDEA StatiCa Connection, bolt bearing is automatically evaluated in the "Bolt Check" tab. It considers bolt type and hole type, edge distance and pitch, and combined tension-shear (if applicable). You can check the bearing utilization per bolt to ensure the lug-gusset interface meets AISC strength requirements.



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)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = \quad 56.75 \ \text{kip} \quad < \quad V = \quad 56.84 \ \text{kip}$$

Where:

$l_c = 0.970$ 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
$t = 1.000$ in	– thickness of the plate
$d = 1.125$ in	– diameter of a bolt
$F_u = 65.0$ ksi	– specified minimum tensile strength of the connected material
$\phi = 0.75$	– resistance factor for bearing at bolt holes

Figure 14 – Evaluation of Limit State of Bolt Bearing in Connection – CBFEM

The value of bolt shear obtained using the AISC equation (56.75 kips) is similar to the value obtained from CBFEM (56.75kips) for the most highly stressed bolts, where 100% bolt utilization is reached under an axial tension load of 345 kips, as shown in Figure 14.

9.e. Bolt Tearout

Bolt tearout (also known as edge rupture) is a critical bolt-related limit state that must be checked in bolted Chevron BRB connections, particularly at the lug-to-gusset plate interface. It occurs when insufficient edge distance allows the bolt to tear a strip of the connected plate under load. This limit state applies to:

- The lug plate, which transfers axial brace load to the gusset via bolts, and
- The gusset plate, which may be thinner and more susceptible to tearout, especially when high axial loads are transferred through few bolts or with tight spacing near edges.

AISC 360-16, Section J4.3 applies to failure along the edge of the connected material due to excessive tension at bolt locations. Bolt tearout must be checked separately for both the lug plate and the gusset plate to ensure minimum edge distances, as specified in AISC Table J3.4.

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

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

While bearing governs local deformation, tearout poses a risk of sudden rupture. Both must be evaluated independently for all connected plates to ensure a safe and code-compliant connection.

In IDEA StatiCa Connection, bolt tearout (edge rupture) is not explicitly checked as a separate limit state like it is in AISC. Additionally, IDEA StatiCa does not automatically flag tearout failure, so it is our responsibility to ensure that edge distances comply with AISC Table J3.4. This verification should be performed by combining IDEA StatiCa's visual results with hand calculations for full confidence.

To assess potential tearout:

- Look for high plastic strains at bolt edges — this may indicate a tearout risk.
- Check for high bolt utilization values (typically >90%), focusing especially on bolts closest to plate edges.

Overall, the limit states of bolt tearout as per CBFEM was observed to agree to the values as per AISC for the presented connection.

9.f. Weld Connections

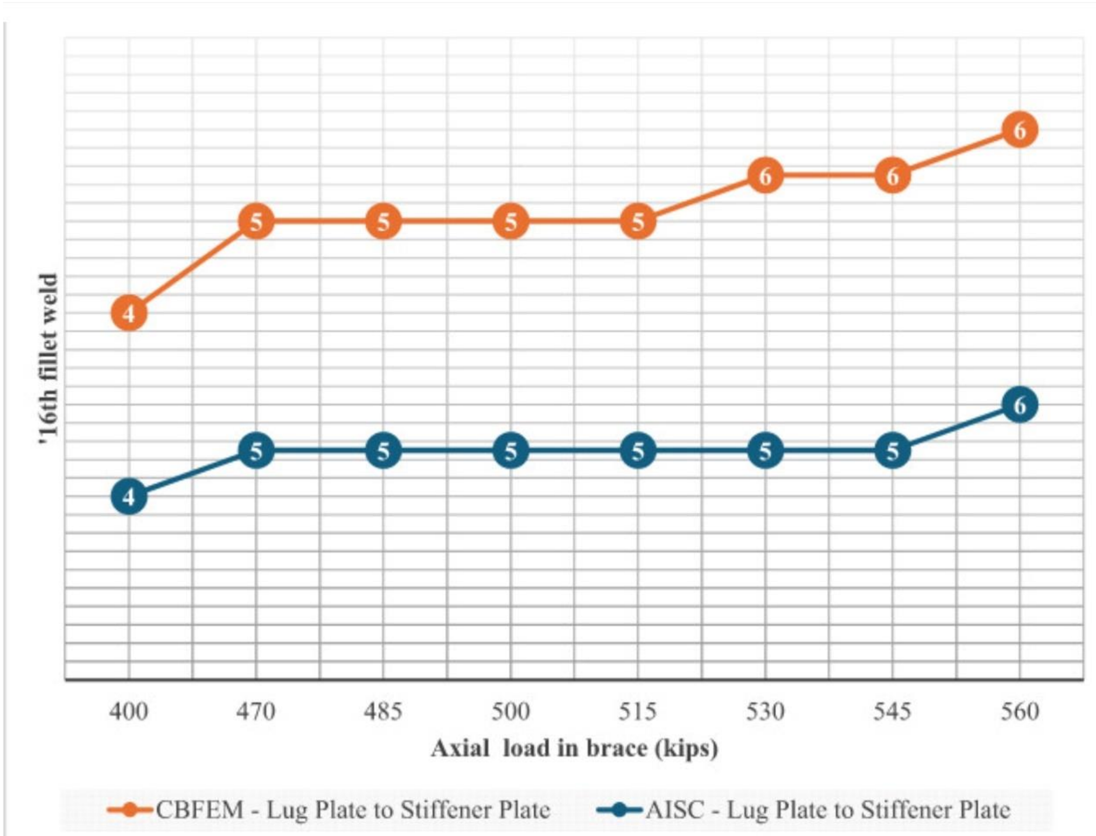
Welds play a critical role in transferring brace forces through various components of BRBF, particularly at lug plates, gusset plates, stiffeners, and beam flanges. Proper weld design ensures that axial, shear, and bending forces are effectively transmitted through the connection without premature failure. Welds must resist high cyclic loads under seismic conditions—fatigue and ductility are critical considerations. Shop welds are preferable for critical elements like lug plates, while field welds must be carefully inspected.

The distribution of forces in a welded connection may be considered either elastic or plastic. An ideal elastic-plastic model is used in CBFEM, where the stress in the weld throat section governs the plasticity state. Weld limit state checks in CBFEM differ from traditional AISC 360-16 design methods. Traditional calculations often neglect small eccentricities, deformations, torsions, and Poisson effects. In contrast, CBFEM includes all these parameters, as it is not limited to simple 2D statics and instead models the full plastic behavior of critical weld elements. The weld model in CBFEM reflects actual weld properties, including geometry, stiffness, and ductility. Plastic redistribution is one of CBFEM's strongest advantages for welded connection design. The force, F_n, and weld angle, ϕ are derived from the from stresses $\sigma_\perp$, $\tau_\perp$, $\tau_\parallel$, along with the weld length and effective throat area, as output by IDEA StatiCa's finite element solver.

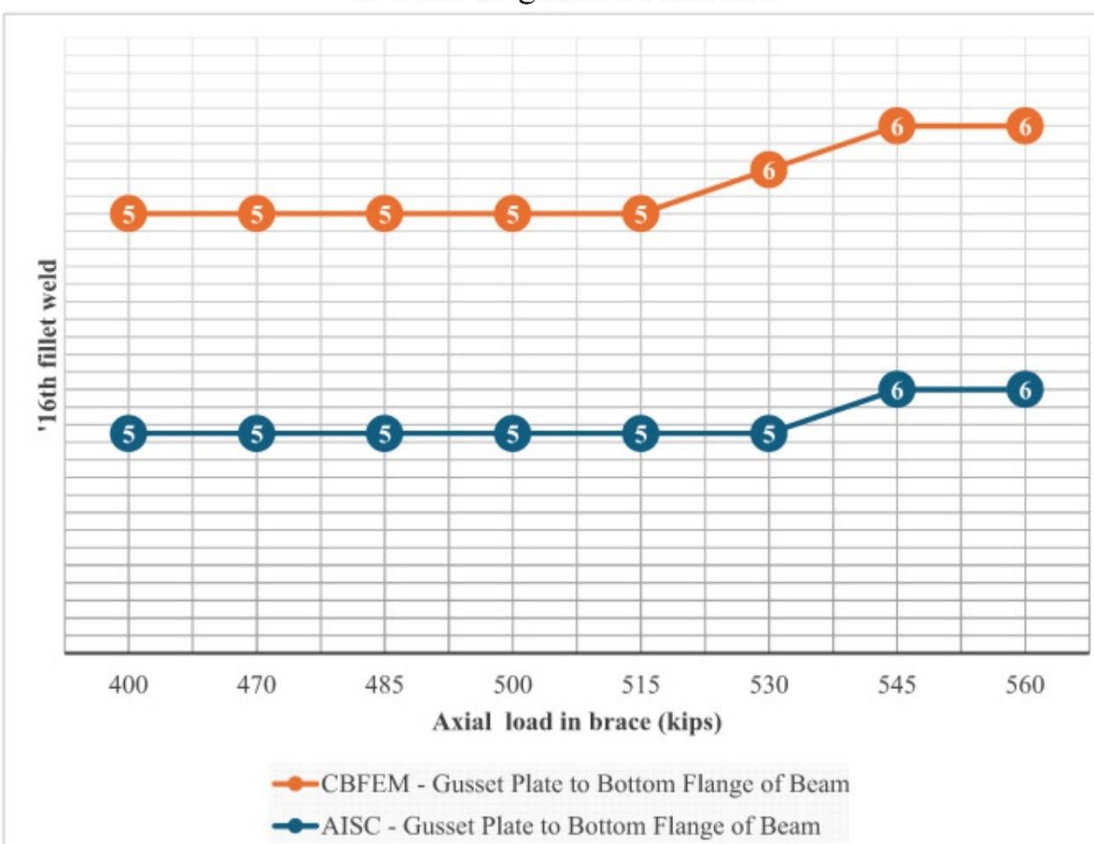
Table 3: Comparison of fillet weld size as per AISC and CBFEM

Sr No.	**P_{brace} (kips)**	**Check of weld connecting**	**Required '16^{ths} weld size (in)**	
			AISC	**CBFEM**
1	405	Gusset plate to beam bottom flange	5/16	5/16
2	510	Gusset plate to stiffener plate	¼	¼
3	560	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.



a) Fillet weld connecting lug plate to stiffener plate
at brace-to-gusset connection



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

Figure 15: Comparison of required size of fillet weld at gusset plate to bottom flange of beam

Figure 15 compares fillet weld sizes required at two key locations in a Chevron BRBF connection—(a) lug plate to stiffener plate, and (b) gusset plate to beam flange—under increasing axial load (400–560 kips). Both CBFEM and AISC 360-16 methods generally agree at lower load levels (≤500 kips), recommending consistent weld sizes (4/16"–5/16"). However, at higher loads, CBFEM predicts a need for larger welds (up to 6/16") due to its ability to capture localized stress concentrations, plastic redistribution, and real load paths, which AISC's simplified elastic approach does not.

The results highlight that while AISC provides conservative and practical estimates, CBFEM offers more accurate, performance-based insight, especially valuable for high-seismic demand connections.

10. Evaluation of Buckling for Compression Load in Brace - CBFEM

In Buckling-Restrained Braced Frames (BRBFs), the key advancement is the brace's capability to undergo yielding in both tension and compression without experiencing global buckling. This behavior is enabled by enclosing a steel core within a buckling-restraining system, which allows the brace to function as a highly ductile, energy-dissipating element during seismic loading. For this mechanism to perform effectively, it is essential that all surrounding components not intended to yield—such as gusset plates, beam flanges, column webs, and stiffeners—remain within the elastic range even as the BRB core yields inelastically.

Among these components, the gusset plate is especially prone to local out-of-plane buckling under compressive forces. Despite the global buckling restraint provided to the BRB core, the gusset plate remains a potential weak link due to its exposure to localized instability. Such buckling not only impairs the brace's energy dissipation capacity but also risks violating the capacity design principles stipulated by AISC 341-16. To address this issue,(Dowswell, Bo, 2006) proposed a mechanics-based approach for evaluating gusset plate compactness. His model treats the gusset as a cantilevered plate subjected to axial compression, providing a rational framework to ensure local buckling is avoided during seismic response.

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:

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

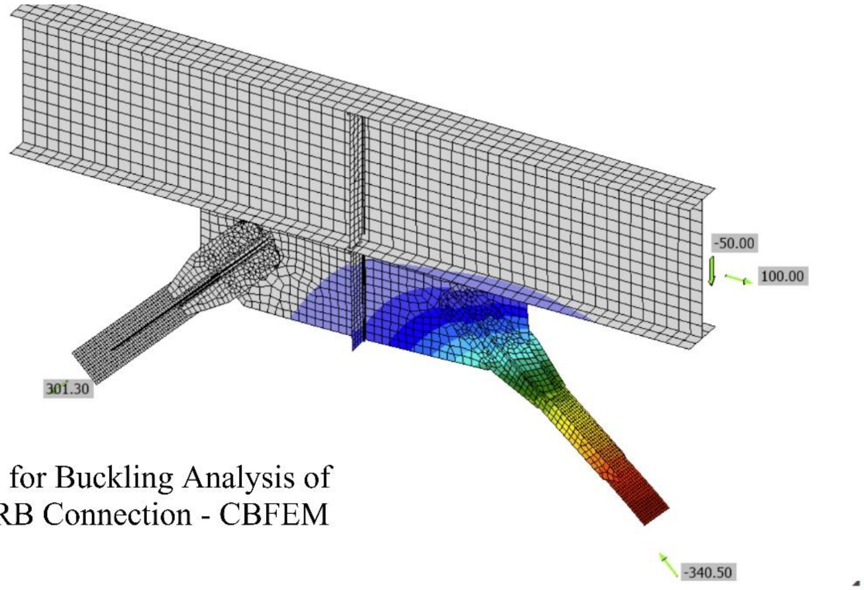
Where,

- ω = Overstrength factor,
- R_y= Material strain-hardening factor (typically ~1.1–1.2),

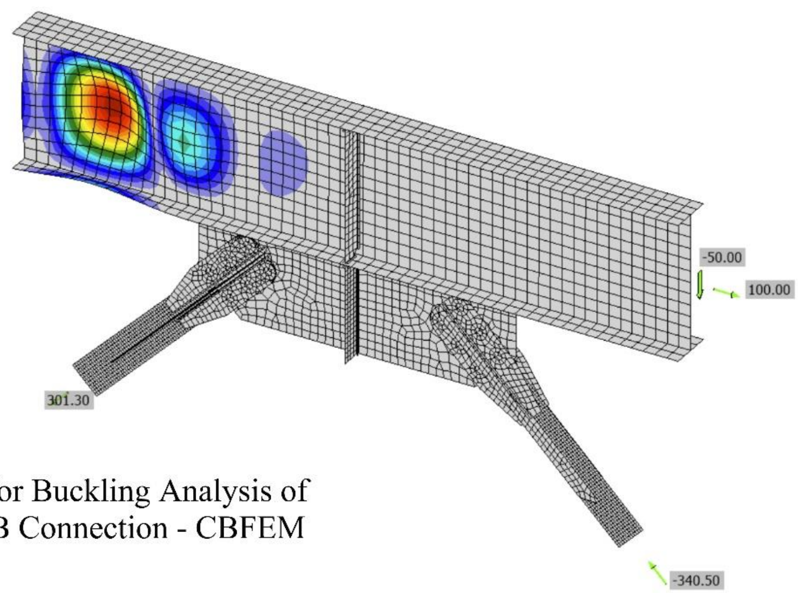
- $F_{y,BRB}$ = Yield strength of steel core in ksi
- A_{core} = Area of the BRB steel core.

The slenderness-based methodology developed by Bo Dowswell has been formally adopted in Appendix D of AISC 341-16, which provides explicit guidance for ensuring gusset plate stability. This includes requirements for minimum effective gusset lengths, limitations on fillet radii and edge distances, and mandates for elastic behavior under expected axial forces.

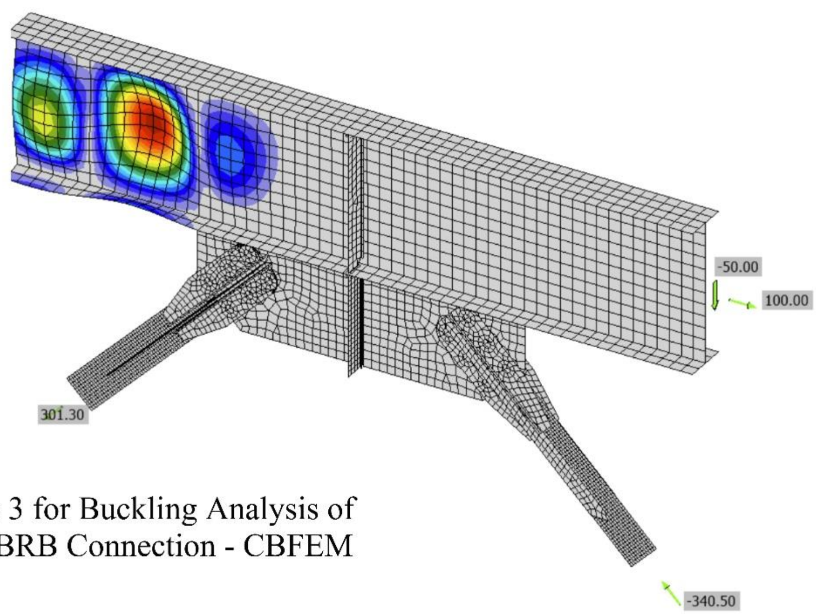
In this study, the gusset plate for the Chevron BRB connection was designed to satisfy these compactness provisions through several targeted strategies. First, the effective length (Le) was minimized by aligning the gusset geometry closely with the brace work points, thereby reducing unsupported lengths. Additionally, 1-inch thick gusset plates were specified to improve local stiffness and buckling resistance. Double-sided lug plates were incorporated to provide additional restraint and promote uniform load distribution. Finally, potential out-of-plane buckling modes of the gusset plate were explicitly assessed using the Component-Based Finite Element Method (CBFEM) implemented in IDEA StatiCa, ensuring compliance with the compactness criteria under seismic demand.



a) Model shape 1 for Buckling Analysis of Chevron Type BRB Connection - CBFEM



b) Model shape 2 for Buckling Analysis of Chevron Type BRB Connection - CBFEM



c) Model shape 3 for Buckling Analysis of Chevron Type BRB Connection - CBFEM

Figure 16 – First Three Buckling Mode Shapes of the Chevron CoreBrace BRB connection - CBFEM

Figure 16 presents the first three buckling mode shapes derived from an eigenvalue analysis of a bolted-lug Chevron BRB connection using CBFEM. Each mode highlights distinct behaviors.

- Mode 1, also referred to as critical buckling mode of the connection shows diagonal flexural buckling of the BRB member, indicating potential lateral-torsional instability under axial compression—a critical check for brace slenderness and gusset design.
- Mode 2 reveals local web buckling of the beam near the gusset, caused by unbalanced vertical forces inherent in Chevron configurations. This emphasizes the need for web stiffeners or doubler plates, if this were to govern design.
- Mode 3 illustrates repeated panel buckling in the beam web, representing higher-order local instabilities which may arise in long-span beams or under cumulative seismic demands.

11. Evaluation of Limit States related to Members – CBFEM

11.a. Web local yielding of beam member

In Chevron-configured Buckling-Restrained Braced Frames (BRBFs), the beam web is subject to concentrated vertical forces resulting from the axial component of the brace force, especially at the gusset plate interface. This introduces the risk of web local yielding, which occurs when bearing stresses from gusset or stiffener plates exceed the yield strength of the web material. This condition is particularly critical when:

- The gusset plate delivers force near the centerline of the web,
- Web stiffeners are absent, limiting load distribution,
- The vertical component of the brace force is large; it is typical with high-strength BRBs.

While AISC 360-16 (Table J10.2) provides equations for evaluating this limit state based on concentrated force and bearing length, IDEA StatiCa's Component-Based Finite Element Method (CBFEM) handles force transfer using distributed shell elements. Consequently, web yielding may not appear as localized in the software output, and no direct utilization check is provided for the web—unlike for bolts or welds. We must instead rely on interpretation of plastic strain and stress distributions to assess the risk of web yielding manually.

Additional complexity arises due to non-uniform load paths, varying brace angles, and diverse gusset geometries, which lead to mixed-mode deformation. Although CBFEM models these effects with high fidelity, it does not align directly with AISC's simplified assumptions regarding bearing zones. Since IDEA StatiCa does not implement AISC-specific checks for web bearing, manual verification using capacity design loads and visual strain contours becomes essential.

For the current connection, web local yielding was evaluated manually using AISC provisions and found not to be a governing limit state, confirming the adequacy of the beam web under the applied seismic loads.

11.b. Web Local Crippling of Beam Member

Web local crippling is a critical limit state characterized by localized crushing and out-of-plane deformation of the beam web under concentrated compressive forces, typically near the flange-to-web junction. This failure mode is more severe and localized than web yielding and can compromise load transfer in the beam, especially when short bearing lengths and the absence of transverse stiffeners intensify stress concentration.

In Chevron-configured Buckling-Restrained Braced Frames (BRBFs), vertical components of brace axial forces are transferred into the beam via gusset plates, often directly into the web near beam flanges. When no stiffeners are present, this concentrated compression may exceed the web's crippling capacity. The risk increases when:

- Brace forces are high; it is typical for seismic BRBs,
- Gusset plate loads are introduced near the web centerline,
- Bearing length is short, and
- Transverse stiffeners are absent.

AISC 360-16 (Table J10.2) provides empirical equations for evaluating web local crippling. If the vertical force exceeds the factored strength (ϕR_n), transverse stiffeners are typically required. However, IDEA StatiCa Connection (CBFEM) does not directly implement these AISC checks. Since loads in CBFEM are modeled as distributed shell contacts, local bearing stresses are smoothed, making critical stress zones less visible—especially when mesh refinement is insufficient.

Moreover, IDEA StatiCa lacks an automatic utilization factor for beam web crippling. Therefore, we must manually evaluate this limit state by:

- Inspecting strain contours near the gusset-to-beam interface,
- Estimating vertical brace forces and bearing zones, and
- Comparing against AISC-calculated web crippling capacities.

Because Chevron load paths often involve complex gusset-to-flange-to-web interactions, and force diffusion due to stiffeners or welds, deformation modes may be mixed. As such, localized stress concentrations may not be explicitly captured in CBFEM, necessitating conservative manual verification.

For the presented case, web local crippling was evaluated using AISC provisions and found not to be a governing limit state.

11.c. Compressive strength of brace member

IDEA StatiCa Connection is primarily developed for the design and code check of joint-level components, focusing on bolts, welds, plates, and stress/strain distribution around connections. However, it is not intended for evaluating member-level axial capacity or global buckling behavior, particularly in Buckling-Restrained Braced Frames (BRBFs), where brace response is complex and governed by nonlinear mechanics. Specifically, IDEA StatiCa does not simulate global brace buckling, yielding behavior along the full brace length, or the effects of transition zones in BRBs. The compressive strength of BRBs depends on factors like the steel core area, material yield strength, strain hardening capacity, and the effectiveness of the restraining casing system—none of which are modeled in IDEA StatiCa. These critical behaviors require full member-level analysis tools that can incorporate unrestrained brace lengths, casing interaction, and inelastic material response.

The Component-Based Finite Element Method (CBFEM) used in IDEA StatiCa assumes simplified boundary conditions at member ends (e.g., bearing or pinned) and focuses solely on local connection response. It does not account for full-length member deformation, slenderness, frame interaction effects, or degradation of axial stiffness under compression. As a result, while IDEA StatiCa is well-suited for verifying connection capacity, it should not be used to assess the overall compressive behavior or buckling resistance of BRBs. That analysis must be performed separately using appropriate global structural analysis tools or manufacturer data.

12. 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	465	3%
2	Block Shear Rupture of Gusset Plate	480	490	1%
3	Bolt Shear	51	51	0%
4	Bolt Bearing	56.75	56.75	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	405	Gusset plate to beam bottom flange	5/16	5/16
2	510	Gusset plate to stiffener plate	¼	¼
3	560	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.

13. Conclusion

This study has thoroughly evaluated the seismic performance and capacity design of a Chevron-configured Buckling-Restrained Braced Frame (BRBF) bolted connection using the Component-Based Finite Element Method (CBFEM), implemented in IDEA StatiCa Connection Version 25.0. Through a systematic assessment of key connection components—brace members, gusset plates, beam flanges, stiffeners, bolts, and welds—the study aimed to verify compliance with AISC 341-16 and AISC 360-16 seismic design provisions and to assess the reliability of CBFEM as a practical analytical tool in modern seismic connection design.

The capacity design approach adopted in this analysis involved incrementally increasing axial loads in the brace members up to 500 kips to examine the onset and progression of critical limit states. The brace members, defined as dissipative elements in accordance with AISC 341-16, exhibited yielding behavior at approximately 365 kips, aligning with the BRBF design philosophy that concentrates inelastic deformation within the brace core. Connection components—such as gusset plates and beam flanges—remained predominantly elastic throughout the load range, verifying their adequacy in transmitting forces without premature yielding.

CBFEM results revealed that tensile yielding and block shear rupture in gusset plates occurred close to their design code predictions, with deviations from AISC calculations within 1–3%, confirming the method's credibility. Similarly, welds were observed to reach full utilization near their design thresholds, with both CBFEM and AISC-based weld sizes agreeing within practical margins. The strain redistribution captured by CBFEM across gusset-beam interfaces and weld seams provided valuable insights into connection ductility and load path evolution, especially under seismic-like loading conditions.

Further investigations into limit states such as web local yielding and web local crippling highlighted certain modeling limitations. IDEA StatiCa does not provide direct checks for these localized phenomena as defined in AISC Table J10.2; hence, manual evaluation was required. However, strain contour plots from CBFEM were effective in identifying regions of concern, even though localized effects may be smoothed due to distributed shell contact modeling.

The vulnerability of the gusset plate to local out-of-plane buckling was addressed using Bo Dowswell's slenderness criterion (Dowswell, 2006), codified in Appendix D of AISC 341-16. The gusset plate was modeled and detailed for compactness using a 1-inch thickness, minimized effective length (L_e), and double-sided lug connections to enhance restraint and stiffness.

To validate these design strategies, an eigenvalue buckling analysis was performed using CBFEM, revealing three critical buckling mode shapes. The first mode demonstrated flexural instability along the BRB, indicating the need for appropriate brace slenderness limits and effective lateral restraint. The second mode revealed potential web buckling of the supporting beam under unbalanced Chevron-induced vertical forces, underscoring the importance of beam web stiffening. The third mode, a higher-order panel buckling form, highlighted local instability regions that may arise under cumulative cyclic loading.

Overall, the Chevron bolted BRBF connection evaluated in this study satisfies the seismic performance objectives defined in AISC 341-16. The CBFEM approach implemented in IDEA StatiCa provides a robust and detailed method for assessing connection strength, component strains, and their interactions under seismic loading. While some manual checks remain necessary—especially for web-related limit states and global brace behavior—the tool allows engineers to visualize connection performance and refine design decisions based on localized strain and force paths. This study supports the broader application of CBFEM in the seismic-resistant detailing of BRBF connections.

14. <u>References</u>

AISC 341. (2016). *Seismic Design Manual. American Institute of Steel Construction, Chicago, Illinois.*

AISC 360. (2016). *Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago, Illinois.*

Dowswell, Bo. (2006). *Effective Length Factors for Gusset Plate Buckling | American Institute of Steel Construction*. Https://Www.Aisc.Org/Effective-Length-Factors-for-Gusset-Plate-Buckling. https://www.aisc.org/Effective-Length-Factors-for-Gusset-Plate-Buckling