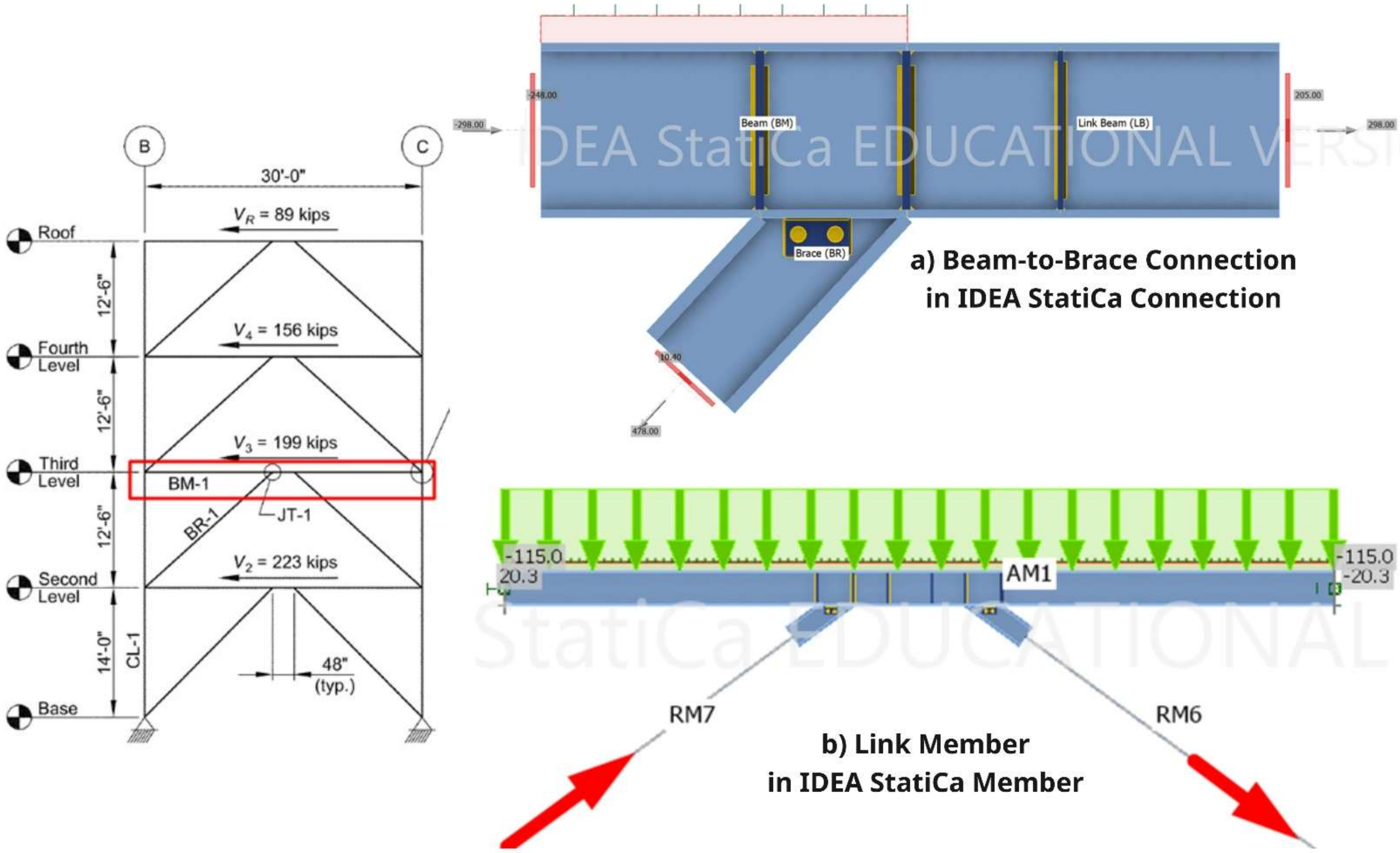
Evaluation of CBFEM for Brace Connection and Link Member in Eccentrically Braced Frame (EBF) System (AISC 360 and 341)



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1. Overview

Eccentrically Braced Frame (EBF) is a seismic lateral force resisting system (LFRS) with high energy dissipation capabilities. Therefore, it is widely used in critical and important structures such as airport control towers, hospitals etc. The behavior of EBF is unique and different from concentrically braced frame (CBF) such as OCBF, SCBF and BRBF, wherein the link in EBF is designated and designed as an energy dissipative element, shown in Figure 1. While in case of CBF, the brace members are designated and designed as energy dissipative elements. The EBF is amongst the highly desirable seismic system, as it combines both high elastic stiffness and high ductility (Berman et al., 2010).

Figure 1 - Link in Eccentrically Braced Frame (EBF) System (AISC 341, 2016)

There are six possible configurations of bracing and link arrangements for eccentrically braced frame as shown in Figure 2 along with their plastic mechanism, wherein the link undergoes yielding.

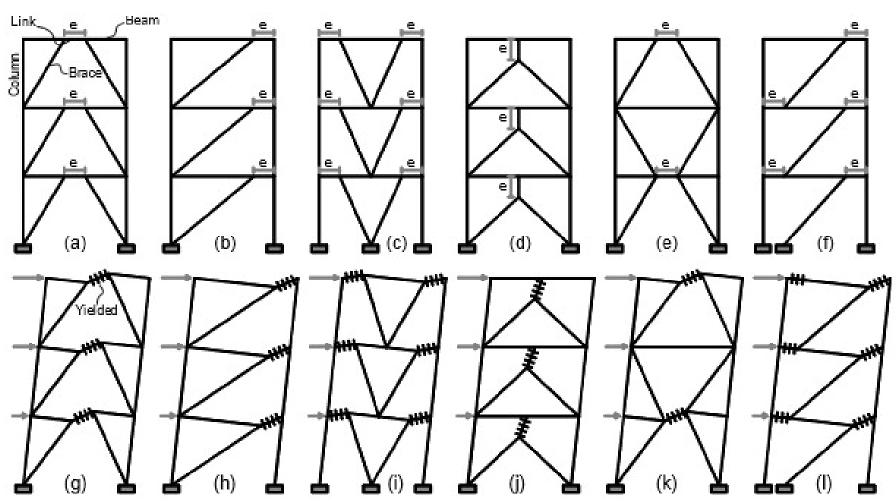


Figure 2 – Typical configuration of members for EBF and their corresponding plastic mechanisms (Bruneau et al., 2011)

Such single bay EBF configurations can position the link horizontally at the ends of the beam as shown in Figure 2 (b), (c), (f), or at the center, as shown in Figure 2 (a), (e) or it can be vertically at center as shown in Figure 2 (d). For links positioned at the ends of the beam, link-to-column connection must conform as per Section F3.6e of AISC341-16. Whereas, for other configurations of link, wherein it is positioned in center of braced frame, the beam-to-column connections must conform as per Section F3.6b of AISC341-16. All the bracing connections in EBF are required to be designed as Section F3.6c of AISC341-16.

The inelastic response in EBF is characterized by a favorable combination of high ductility, stiffness and strength. The energy dissipation is achieved through the yielding of a link for F_x which is lateral seismic force at the considered level, and calculated using Equations 12.8-11 and 12.8-12 of ASCE7-16 as shown in Figure 3, while the other frame members such as beam outside of link, braces and columns should remain essentially elastic (Koboevic & David, 2010).

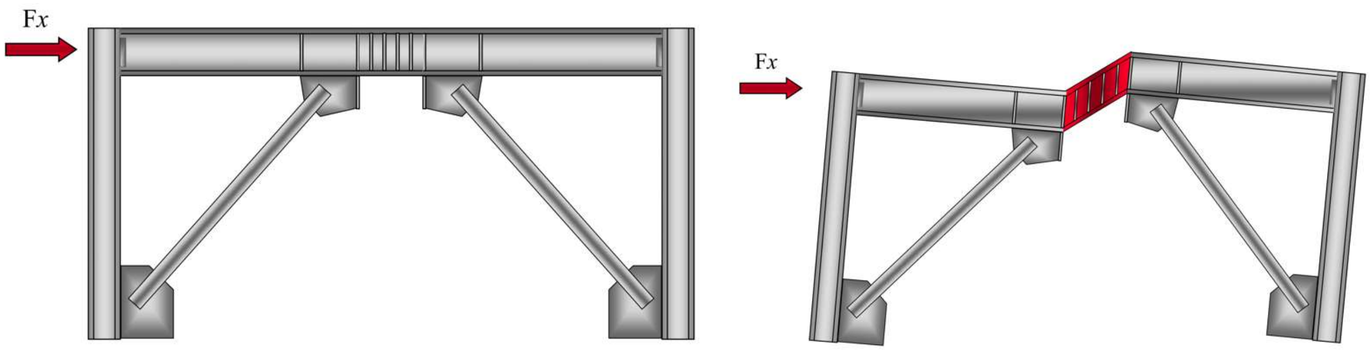


Figure 3 - Inelastic Response of EBF (AISC 341, 2016)

2. Analysis and design requirements for EBF

The analysis and design requirements for EBF are addressed in Section F3 of AISC341-16. The intent of the provisions is to ensure the shear or flexural yielding, and energy dissipation occurs primarily in link only. As given in section F3.3, the required strength of diagonal braces and their connections beams outside the links, and columns are determined using the capacity-limited seismic load effects, which is taken as the forces developed in the member assuming the forces are at the ends of the links and correspond to the adjusted link shear strength.

As noted in AISC 341 Section F3.5(a), the diagonal brace and beam segment outside of the link are intended to remain essentially elastic. Hence, the shear strength of link in accordance with Section F3.3 is "adjusted" to include the amplification effect of both material overstrength and strain hardening. The adjusted shear strength of the link (V_{mh_link}) calculated as $V_{mh\_link} = C_{pr}R_yV_p$

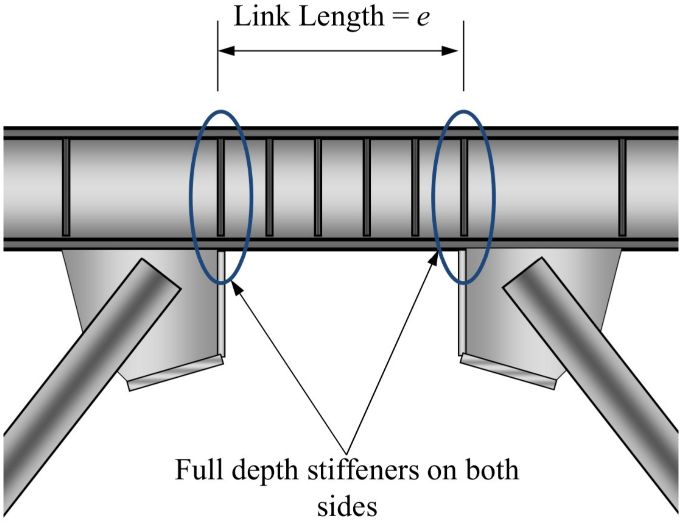
where,

- C_{pr} is a strain hardening factor. 1.25 for I-shaped links (commonly used) and 1.4 for box or built-up links.
- R_y is the ratio of the expected yield stress to specified minimum yielding stress F_y of the material. Values as given in Tabel A3.1 of AISC341-16.
- V_p is the link shear strength determined from Section F3.5b.2 of AISC341-16

The braces and the beam outside of the link shall satisfy width-to-thickness limitations for moderately ductile members. While columns shall satisfy the requirements for highly ductile members. Link shall be an I-shaped cross section (rolled wide-flange section), or built-up box section and must satisfy the requirements of Section D1.1 for highly ductile members. Except, for $e \leq (1.6M_p/V_p)$ flanges of I-shaped links, and webs of all links are permitted to satisfy the requirements for moderately ductile members. The requirements for moderately ductile and highly ductile are given in table D1.1 of AISC341-16.

The beam outside of link, brace and column are designed to resist the loads developed by the fully yielded and strain hardened link. The required strength of beam outside of link (V_{mh_beam}) is based on adjusted shear strength of the link and calculated as $V_{mh\_beam} = 0.88C_{pr}R_yV_p$. The 0.88 factor here accounts for increased strength provided by composite slab and recognizing the fact that limited yielding is unlikely to be detrimental to frame behavior. The axial force in beam outside of link is also based on the adjusted shear strength of the link and given as $P_{Emh} = (V_{mh\_beam} L)/(2H)$, where L is the length of beam in inches and H is the height of level for considered EBF in inches. Additional lateral bracing along the length of the beam, if required, is designed based on Appendix 6 of AISC Specification.

The required strength of the brace in EBF calculated using $V_{mh\_brace} = C_{pr}R_yV_{mh\_link}$ is a combination of factored gravity forces plus the forces generated by adjusted link shear strength, using the load combinations that includes the overstrength seismic load, as per Section F3.3 of AISC 341-16.



a) Typical design detail

b) Actual image of link beam and connection

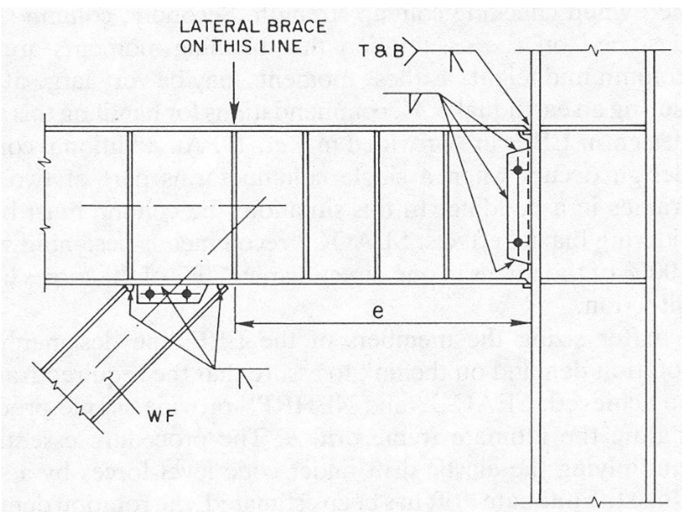
Figure 4 - Stiffeners in link of EBF (AISC 341, 2016)

Full depth, double-sided web stiffeners as shown in Figure 4 satisfying the minimum required width and thickness as per Section F3.5b(4) of AISC 341-16 are required at the diagonal brace ends of each link for case of I-Shaped cross section of link. The use of same stiffener thickness is preferred for the intermediate stiffeners, to simplify the detailing and fabrication. Intermediate stiffeners in link are required for all conditions and must be full depth, except when $e > 5M_p/V_p$, where e is link length in inch, M_p is plastic bending moment of a link in kip-in and V_p is plastic shear strength of link in kips as per Section F3.5b(4) of AISC 341-16. Similarly, the required strength of fillet welds connecting a link stiffener to the link web shall be F_yA_{st}/α_s, whereas for fillet welds connecting the stiffener to the link flanges is $F_yA_{st}/(4\alpha_s)$, where F_y is specified minimum yield stress of the stiffener, A_{st} is the horizontal cross-sectional area of the link stiffener and α_s is the LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD as per Section F3.5b(4) of AISC 341-16. HSS sections are not permitted to be used as link in EBF

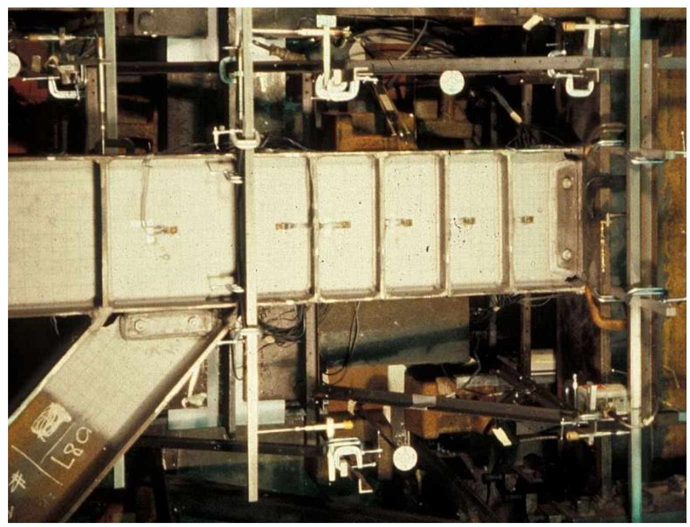
In terms of economy, when considering both the plate material and fabrication, the cost of built-up shapes as beam members in EBF (link and beam outside of link) can be three to four times to that of a rolled wide flange section. There, built-up I-shaped link sections are preferred only when (V_{mh}/V_r) capacity ratio for a particular level of building with EBF exceeds 2, where V_r is the design shear force of link in kips(*2021 IBC SEAOC Structural/Seismic Design Manual, Volume 4: Examples for Steel-Framed*, n.d.). The link stiffener requirements for built-up box section are discussed in Section F3.5b(5) of AISC 341-16.

As the link serves as an energy dissipative item in EBF, it is designated to be a protected zone and is required to meet the requirements of Section D1.3 as per Section F3.5C, such that headed stud anchor, doubler plates, penetrations, concrete slabs are amongst the items not permitted. Whereas the beam segments outside of the links are composite with the concrete slabs. The beam-to-column connection to satisfy the requirements of Section F3.6b. The gusset plates in SCBF are required to accommodate the inelastic rotation due to brace buckling, whereas in case of EBF, as the brace is not the structural fuse. The user notes in AISC 341-16 in Section F3.5a indicate that brace-to-gusset connection be designed to resist the adjusted brace strength corresponding to link yielding and strain hardening in accordance with ordinary concentric braced frame requirements as outlined in Section F1.6. Column splices must comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration (CJP) groove welds.

The link-to-column connection as shown in Figure 5 are similar to beam-to-column connections of SMF and must conform to various requirements as stipulated in Section F3.6e. Connection performance must be verified by large-scale testing following the requirements of Section K2, or a connection must be used that has been very thoroughly tested and reviewed so as to be deemed prequalified in accordance with the requirements of Section K1.



a) Typica design detail



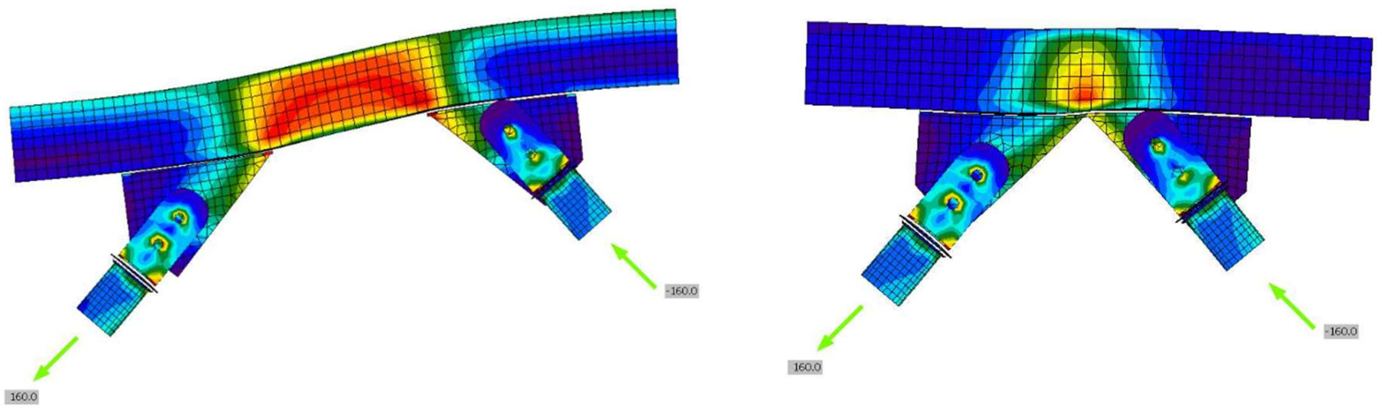
b) Experimental testing of design detail

Figure 5 - Link-to-column connection in EBF (AISC 341, 2016)

To provide a resilient seismic response, EBF is required to meet several quality requirements as outlined in Chapter J of AISC 341-16 and Chapter N of AISC360-16.

3. Comparison with concentrically braced frame (CBF)

The design of connections in EBF is very similar to that of SCBF with some exceptions. First, the dissipative item which is designed based on the capacity method in case of SCBF is the brace member, while for the EBF it is the link. Therefore, the energy dissipation in EBF takes place through the deformation of the link where plastic hinge is formed as shown in Figure 6, and to some extent is found to behave better than concentrically braced frame, provided the connections are properly designed and detailed for the action of seismic loads.



a) Eccentrically Braced Frame (EBF) b) Concentrically Braced Frame (CBF)

Figure 6 – Behavior of EBF when compared to CBF (*IDEA StatiCa UK*, 2022)

Since the brace is not a dissipative item in EBF, it is not expected to yield and therefore, providing the 2t buckling link in the gusset plate is not preferred. Due to the presence of eccentricity of the bracing members in EBF, this creates a large bending demand for the link which is resisted by the beam outside of link and by the bracing member. Due to the higher R factor for EBF than other braced systems, there is a substantial reduction in seismic base shear force, which may result in savings in the construction of the diaphragm and foundation.

4. Problem Description

The objective of this example is to verify the component-based finite element method (CBFEM) for a brace-to-link connection as well as member in four story configurations of eccentrically braced frame (EBF) for a steel building. The check of brace-to-beam connection node is performed in IDEA StatiCa Connection, while IDEA StatiCa Member is used to evaluate the stability and buckling of link in EBF.

The floor plan details of considered building, along with the connection details as provided in Example 5.4.6 for EBF in AISC341-16 are considered as presented in Figures 7 and 8, respectively. The results obtained from calculation method which are based on Steel Construction Manual, Fourteenth Edition (AISC 360, 2016) and Seismic Design Manual, Third Edition (AISC 341, 2016) are then compared with results obtained from the CBFEM using "capacity design" analysis in IDEA StatiCa version 24.1. The applicable building code specifies using ASCE/SEI 7 for calculating the loads.

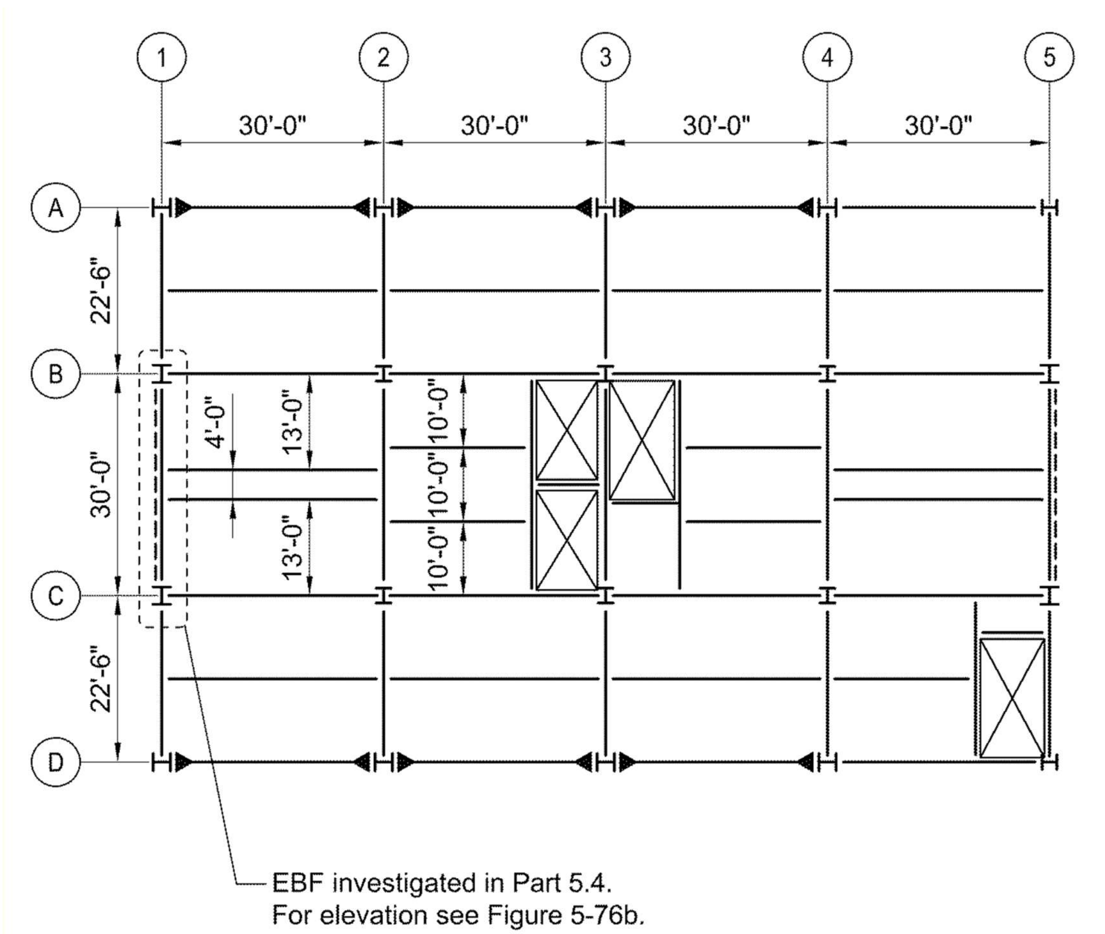


Figure 7 - Floor plan for steel building highlighting considered EBF (AISC 341, 2016)

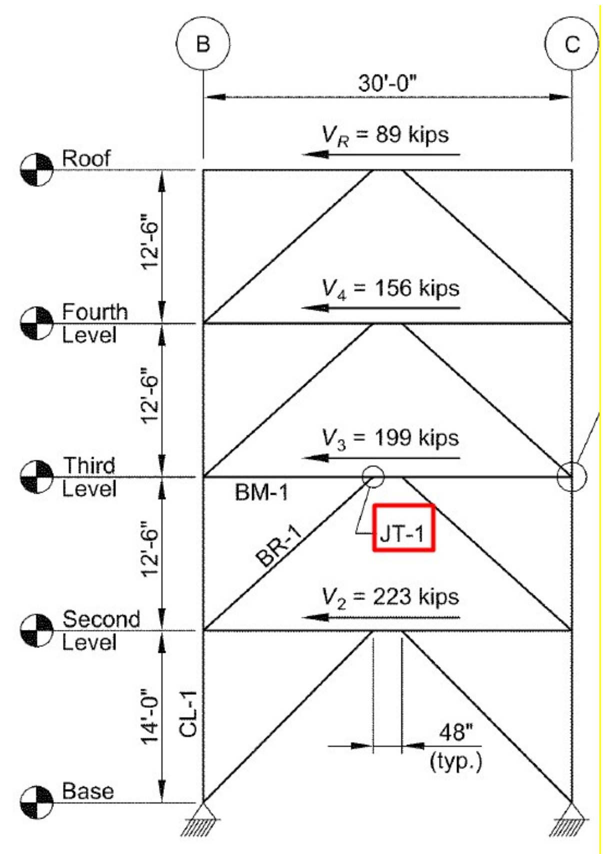


Fig. 5-76b. EBF elevation.

a) EBF elevation (AISC 341, 2016)

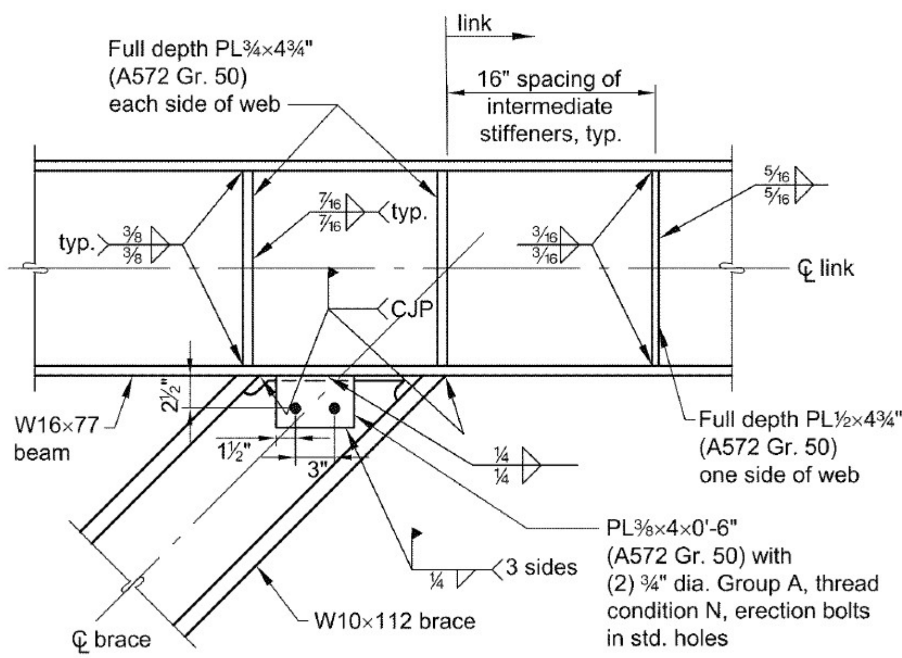


Fig. 5-78. Connection as designed in Example 5.4.6.

b) Connection design details for brace-to-link in EBF (AISC 341, 2016)

Figure 8 - Considered connection for evaluation in CBFEM (AISC 341, 2016)

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

Member Details

1. **Link cross-section**

- W16x77
- ASTM A992

2. **Beam outside of link cross-section**

- W16x77
- ASTM A992

3. **Brace cross-section**

- W10x112
- ASTM A992

Plate Details

1. **Stiffening Plates (Brace to link)**

- PL3/8”x4”x6”
- ASTM A572-50

2. **Stiffener Plates**

- Full depth PL3/4”x4-3/4” at each side of beam web
- ASTM A572-50

Connection Details

1. **Erection Bolts (Only at shear plate of brace)**

- Group A Bearing Type Bolts (bolts in standard holes) with threads included from the shear plane (thread condition N),
- 3/4” diameter bolts; A325-N

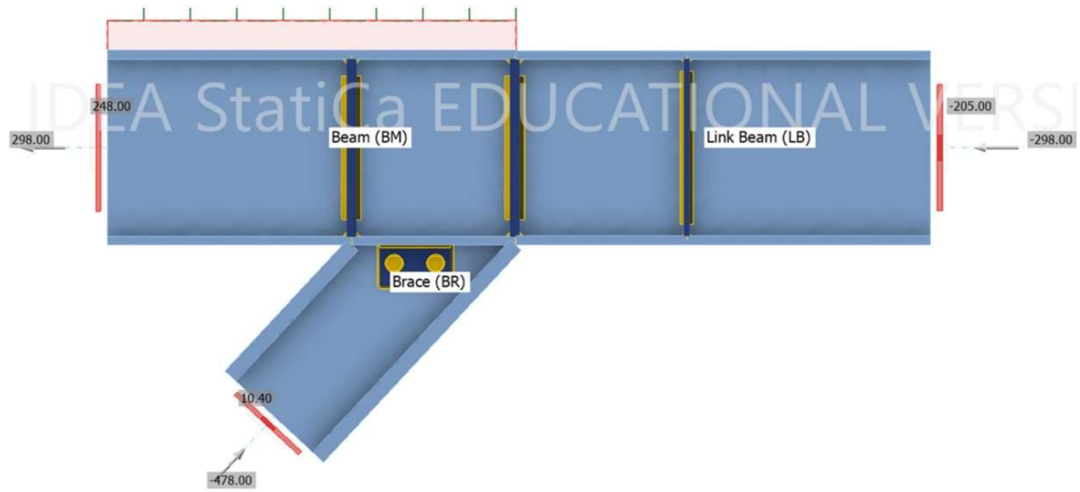
2. **Welded Connections**

- E70xx electrode
- 1/4” rear sided fillet weld between shear plate on brace web to bottom flange of link beam
- 1/4” rear sided fillet weld connecting brace web to shear plate.
- For stiffeners in link beam web near the brace-to-link connection, use 3/8” double sided fillet weld to connect beam flanges to stiffeners, and 7/16” double sided fillet weld to connect beam web to stiffeners.
- For stiffeners in link beam web away from the brace-to-link connection, use 3/16” double sided fillet weld to connect beam flanges to stiffeners, and 5/16” double sided fillet weld to connect beam web to stiffeners.
- CJP groove welds to connect the brace flanges to the beam flange.

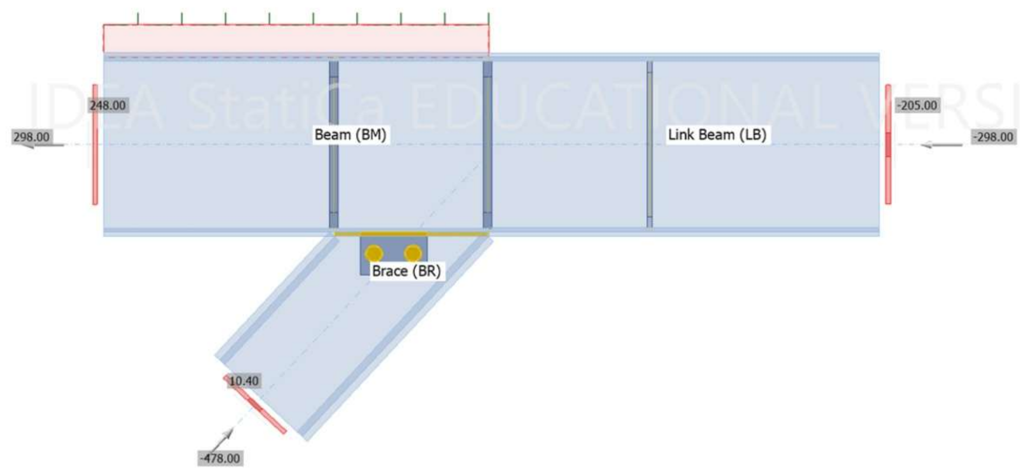
5. Modelling and analysis of brace-to-beam connection

The connection modelling in IDEA StatiCa begins with defining the members - link, beam outside of link and brace member. Then, the two stiffening plates on brace web connecting to bottom flange of beam and five stiffener plates in beam web are defined and arranged as per connection details using several manufacturing operations. The brace is oriented with the web in the plane of the frame; such that erection bolts are used to connect stiffening plate in the web of the brace to the bottom flange of the beam by fillet welds. Lateral torsional restraint is modelled in CBFEM for the top flanges of the beam outside of link using the manufacturing operation in IDEA StatiCa Connection.

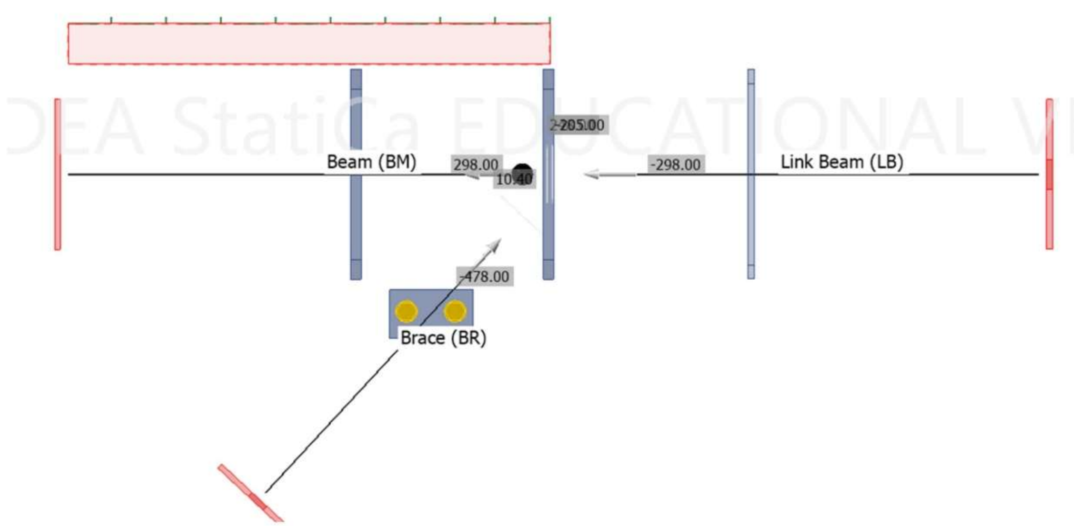
The solid, transparent and wireframe view of the base model is shown in Figure 6. The beam outside of link is selected as a bearing member in CBFEM and the model type of $N\text{-}V_y\text{-}V_z\text{-}M_x\text{-}M_y\text{-}M_z$ (Fixed) is assigned to it. Similarly, model type of $N\text{-}V_y\text{-}V_z$ (Pinned) is assigned to diagonal brace and link individually, such that the forces are in the node, as seen in wireframe view in Figure 9.



a) Solid view of EBF Brace Connection - CBFEM



b) Transparent view of of EBF Brace Connection - CBFEM



c) Wireframe view of EBF Brace Connection - CBFEM

Figure 9 - Configurations of the view of the connection for EBF – Solid, Transparent and Wireframe view

According to AISC 341-16 Section F3.3, the required strength of diagonal braces and their connections, beam outside links, and columns shall be determined using capacity-limited horizontal seismic load effect, E_{cl} and based on the forces developed in link assuming the forces at the ends of the links correspond to the adjusted link shear strength. For the case of eccentrically braced frame, link is a dissipative element and are expected to undergo significant inelastic plastic deformation for the action of governing seismic load case. They provide excellent energy dissipation and higher ductility to the entire system. In CBFEM, when performing capacity design analysis for connections in eccentrically braced frame, the links are required to be selected as "dissipative items" as shown in Figure 10.

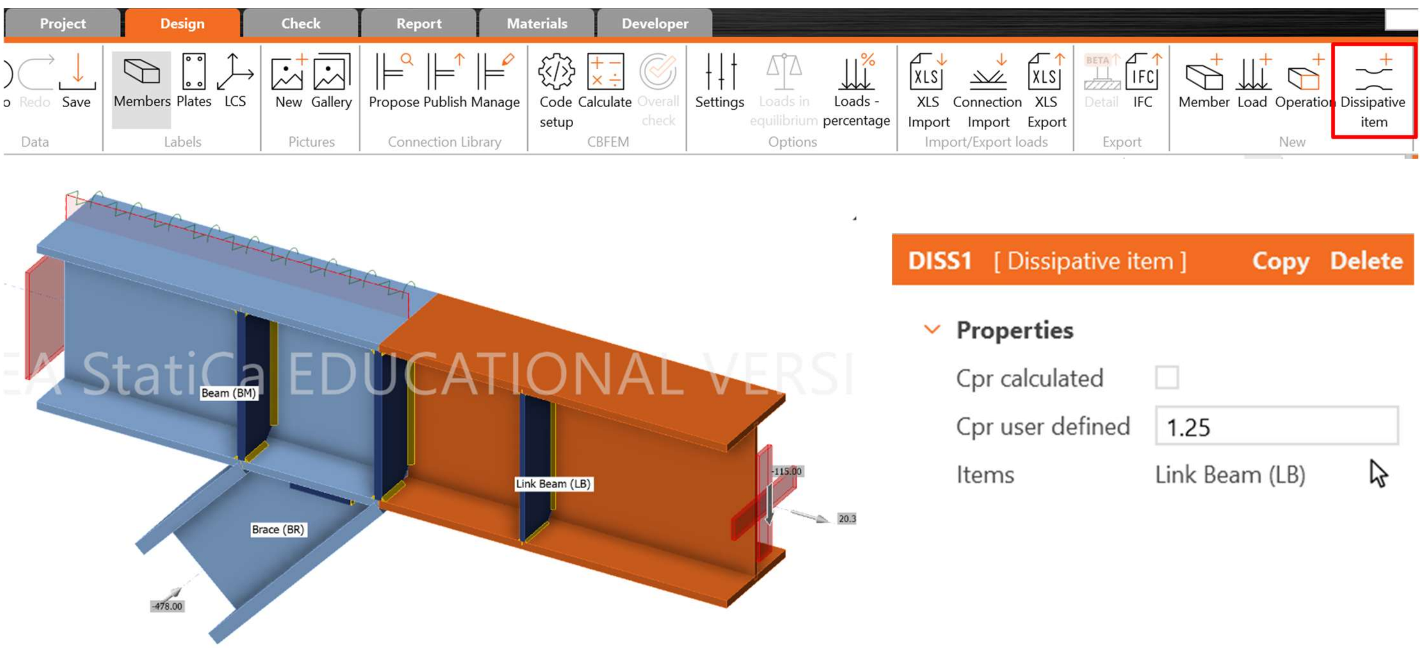


Figure 10 – Link as "dissipative items" for EBF in Capacity design analysis - CBFEM

Three factors are assigned to the yield strength of the dissipative item: R_y, R_t and C_{pr}. R_y, which is the ratio of probable to minimum yield strength given in Table A3.1 of AISC341-16, C_{pr}, strain hardening factor, is assigned a value of 1.25 for the case of I-Shaped link. Lastly, the ultimate strength of the dissipative item is increased by factor R_t which is the ratio of probable to minimum tensile strength given in Table A3.1 of AISC341-16. When the analysis is running, the material strength of dissipative items is increased by the overstrength factor (R_y), and strain hardening factor (C_{pr}) as shown in Figure 11.

Design data

	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

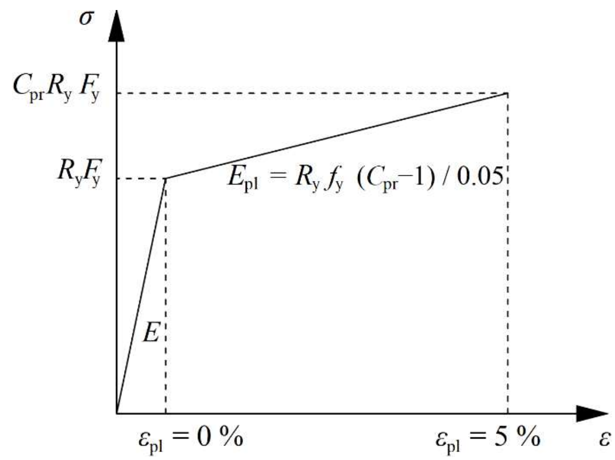
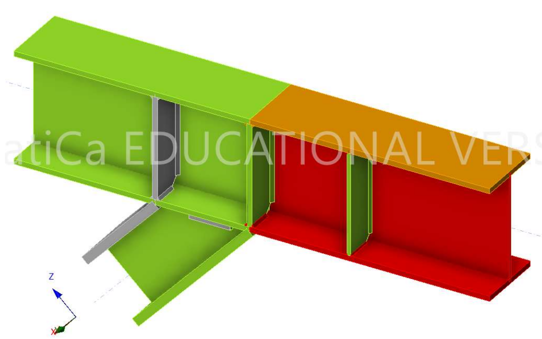


Figure 11 – Yield strength factors for capacity design analysis and stress-strain diagram - CBFEM

Visualization of connection or member failure can be easily predicted in CBFEM using the **"Traffic light type format" of overall check** as obtained from the CBFEM after running the analysis. Additionally, details are given in Figure 12 and Table 1,



Visualization of the overall check in CBFEM
(Traffic light type format)

1. **GREY** – Represents those elements which, although could be working well, might be beneath the optimum range (**60%**) of percentage utilization, and therefore will be safe.
2. **GREEN** - Represents the elements, which have a utilization percentage between **60% to 95%.**
3. **ORANGE** – Represents components, that despite not exceeding the maximum capacity, are working at the limit, i.e. **between 95% and 100%.**
4. **RED** – Represents the components that does not satisfy checks and have a utilization **beyond 100%.**

Figure 12 - Visualization of failure in connection and member in overall check – CBFEM

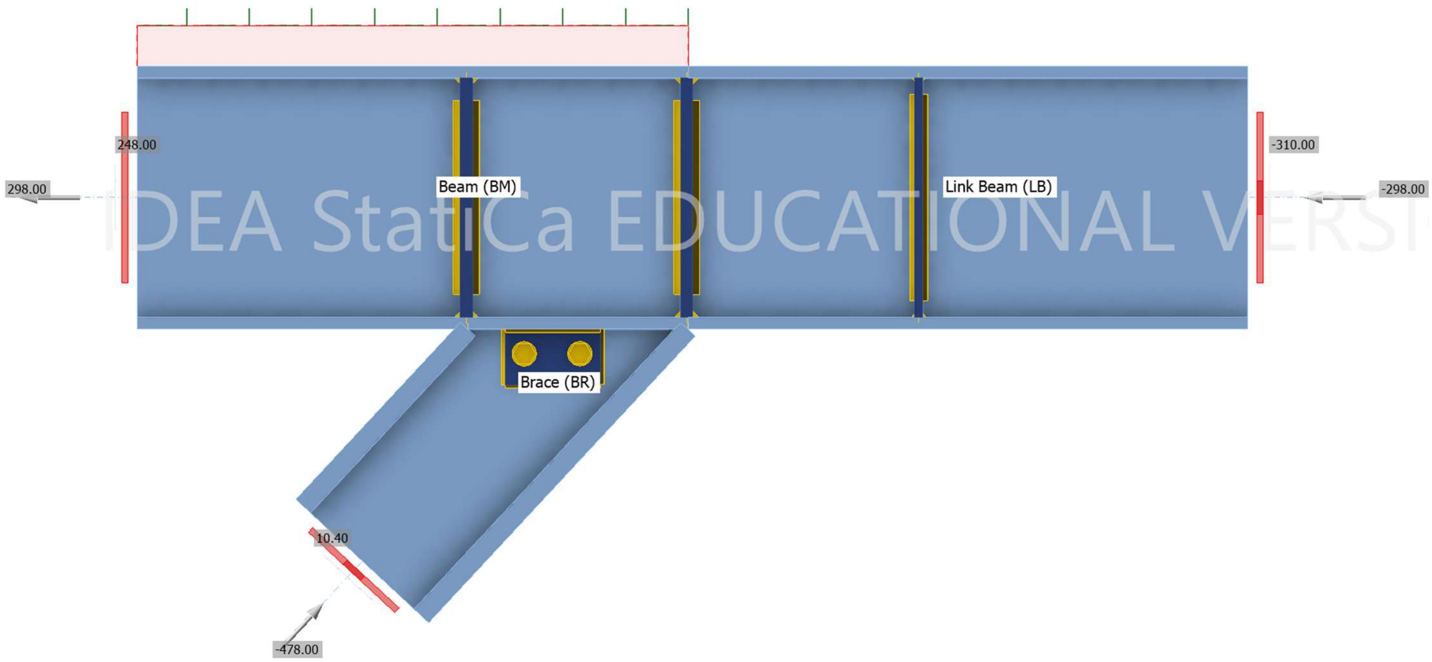
Based on the observation for the given connection from Figure 5, conclusions are tabulated in Table 1 below,

Sr No	**Color**	**Elements**	**Comments**
1	**GREY**	Two stiffeners in beam outside of link, top flange of brace etc.	Elements are **safe for action of loads.**
2	**GREEN**	Web and Bottom flange of brace, beam outside of link etc.	Elements are **safe for action of loads**.
3	**ORANGE**	Top flange of link etc.	Elements are **on the verge of failure**, for the applied loads.
4	**RED**	Web and bottom flange of link, weld at several locations, etc.	Elements are failing for the applied loads; utilization is **beyond 100%.**

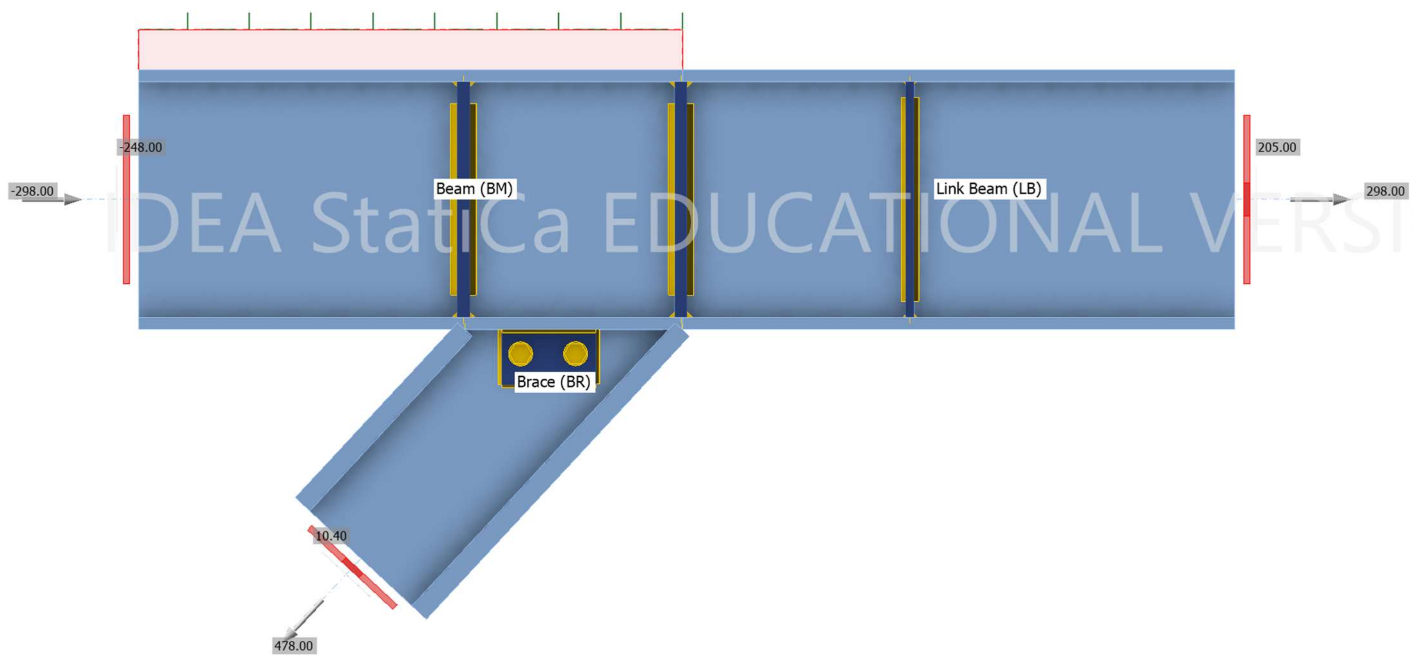
Table 1 – Traffic light type format visualization of the connection in CBFEM

6. **Evaluation of brace-to-beam connection in CBFEM**

The AISC 360-16 (LRFD) as a design code and capacity design as analysis method is used in CBFEM. The connection is subjected to the action of design loads in members meeting at a joint, as obtained from analysis (shown in attached appendix in the end) as shown in Figure 13, and the CBFEM evaluation is performed using IDEA StatiCa Connection V24.1.



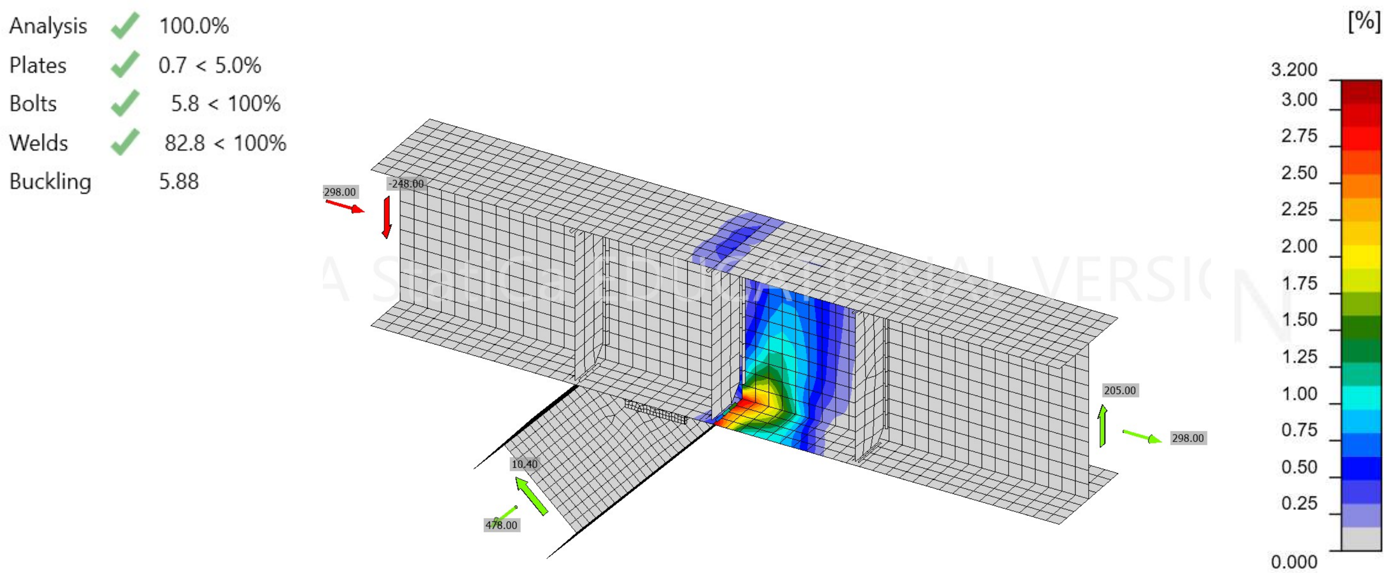
a) Design load case 1 – Compression load in brace



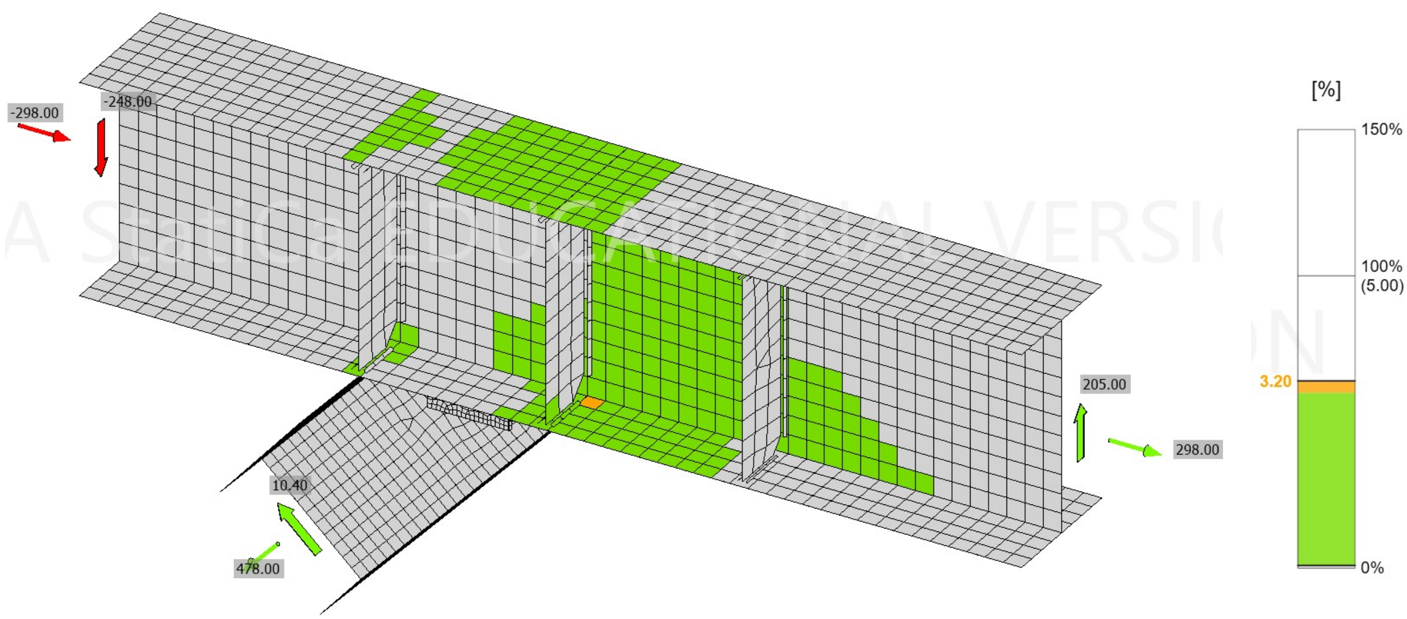
b) Design load case 2 – Tension load in brace

Figure 13 – EBF brace-to-link connection in IDEA StatiCa Connection - CBFEM

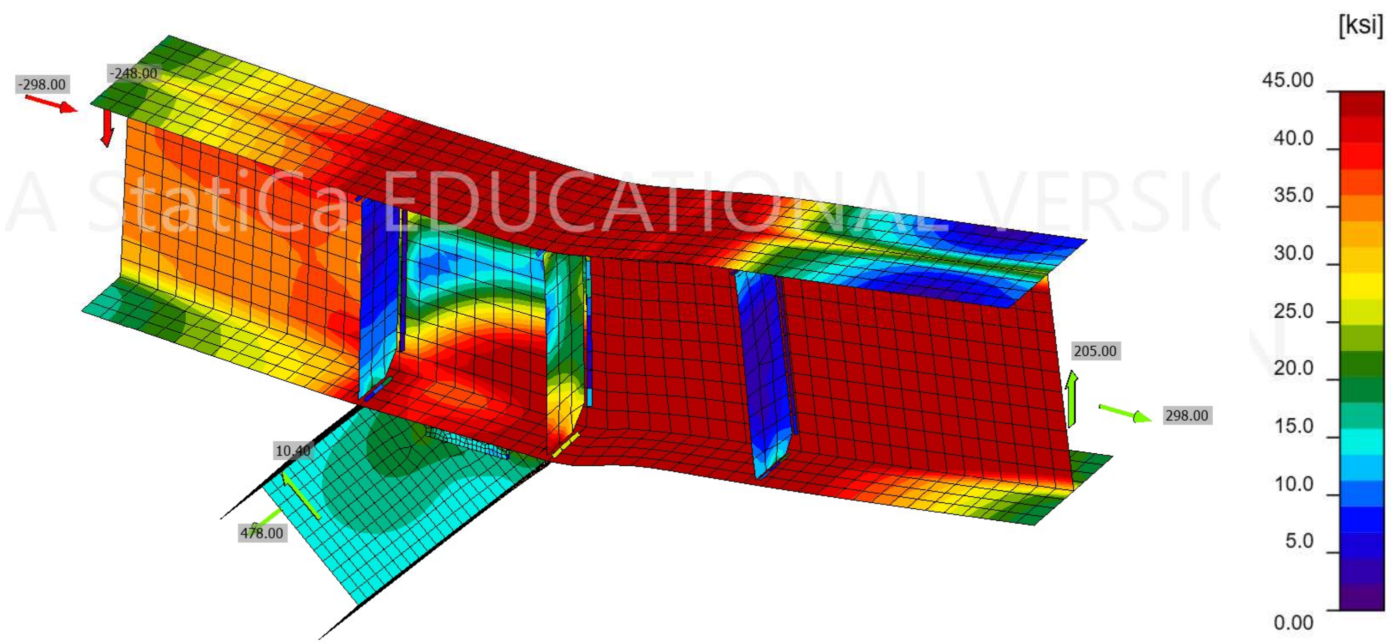
Overall, the connection (plates, bolts and welds) is found to be safe for action of the design loads as observed from the analysis results and shown in Figure 14.



a) Plastic strains (%) in EBF Connection - CBFEM



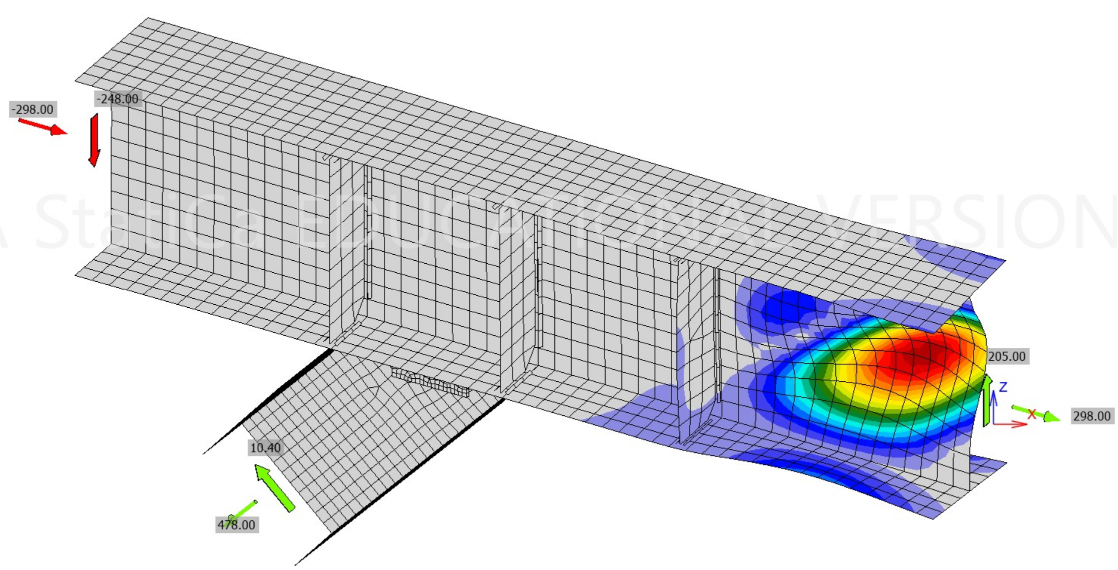
b) Strain check (%) in EBF Connection - CBFEM



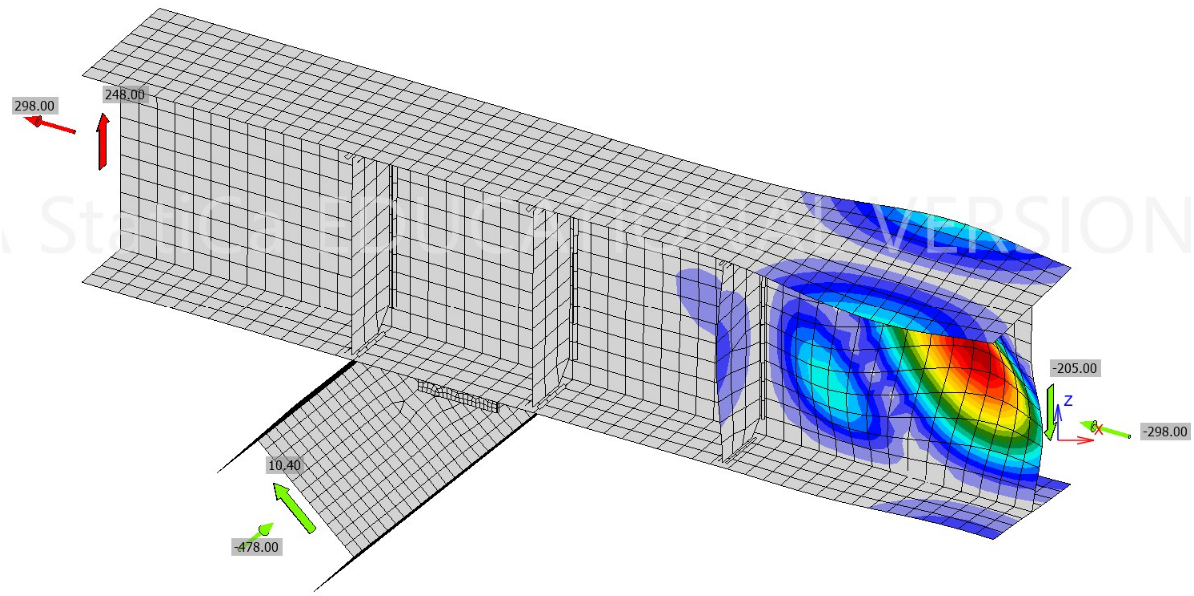
c) von Mises stresses (ksi) in EBF Connection – CBFEM

Figure 14 –Evaluation of brace-to-beam connection for design loads in EBF - CBFEM

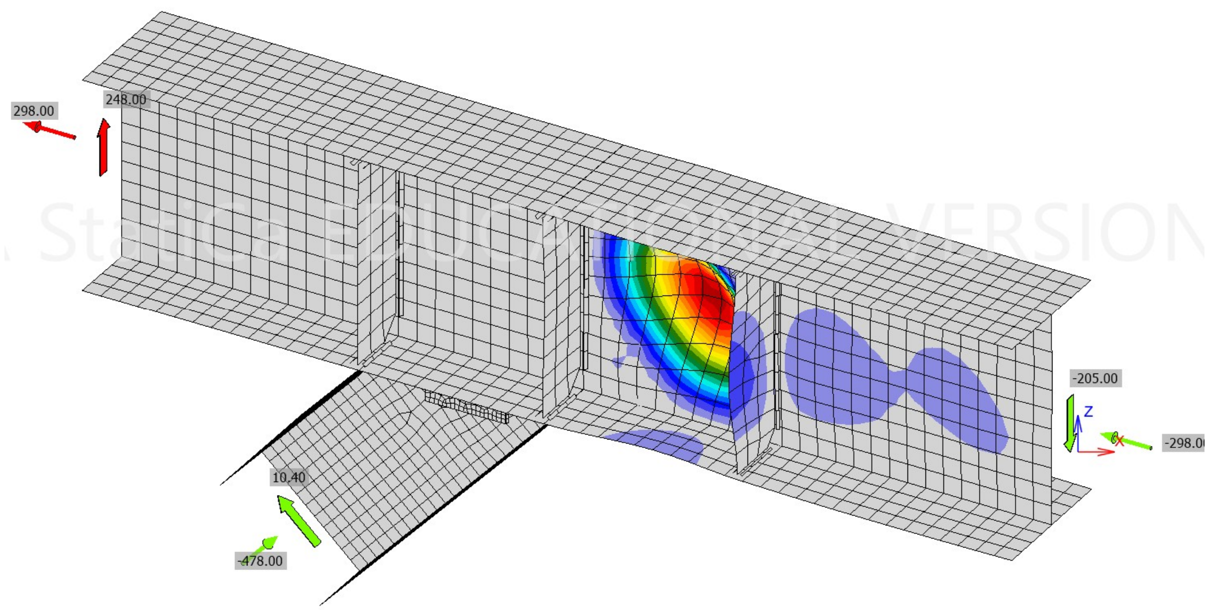
IDEA StatiCa Connection allows users to perform linear buckling analysis. The result of linear buckling analysis is elastic critical buckling factor (α_{cr}) corresponding to the buckling mode shape for the design compression load in brace. Based on the buckling analysis for the given connection, the first mode of buckling is observed in the web of the link as shown in Figure 15, and the elastic critical buckling factor is obtained as 7.8 from CBFEM. This type of buckling can be classified as local buckling, as the brace is constraint by one side to the member. Since the critical buckling factor value of 5.88 is observed to be more than the minimum required critical buckling factor value of 3 for local buckling, the connection is safe for buckling as per CBFEM. Similarly, buckling mode shape 2 and 3 are observed in the web of link.



a) Buckling mode shape 1- CBFEM (Buckling factor = 5.88; Deformed shape factor = 10)



b) Buckling mode shape 2 – CBFEM (Buckling factor = 5.92; Deformed shape factor = 10)



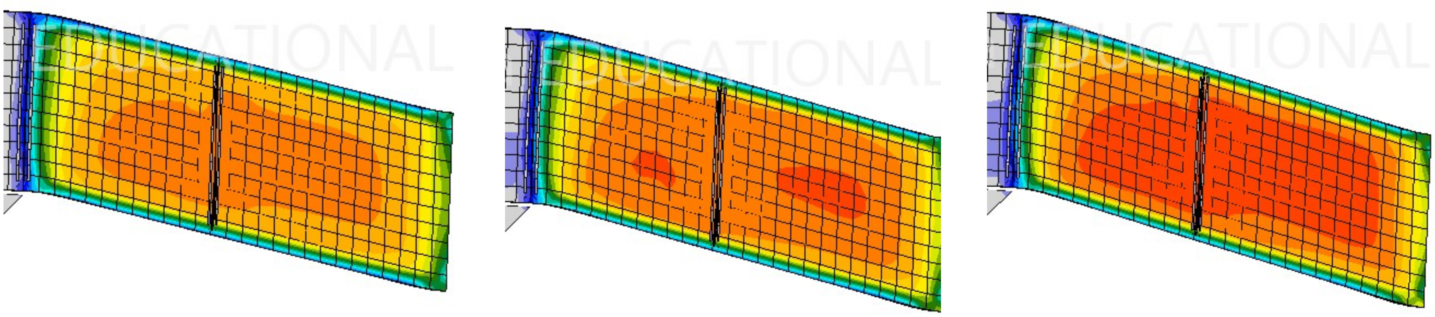
c) Buckling mode shape 3 – CBFEM (Buckling factor = 7.7; Deformed shape factor = 10)

Figure 15 – Buckling Analysis of brace-to-beam connection in EBF – CBFEM

To evaluate the behavior of link for the increased brace loads and to obtain loads on brace and link such that the plastic strain in the plates is around 5%, the design brace load of 478kip was increased by 20kips each – 498kip, 518kip and 538kip, and the strains in the link are obtained, before a plastic strain of 5% is reached in connection as shown in Table 2 and the behavior is seen in Figure 16.

Load Case	**Brace Member**	**Link**		**Maximum Plastic Strain – IDEA StatiCa Connection (CBFEM)**
	Axial Load (kips)	**Axial Loads (kips)**	**Vertical Shear (kips)**	
1	-498	169.5	-364.8	3.7% < 5%
2	-518	176.3	-379.5	4.3% < 5%
3	-538	183.1	-394.1	4.9% < 5%

Table 2 – Evaluation of connection for increased loading in brace member – CBFEM



a) Load Case 1 – CBFEM (Deformed shape factor = 1)

b) Load Case 2 – CBFEM (Deformed shape factor = 1)

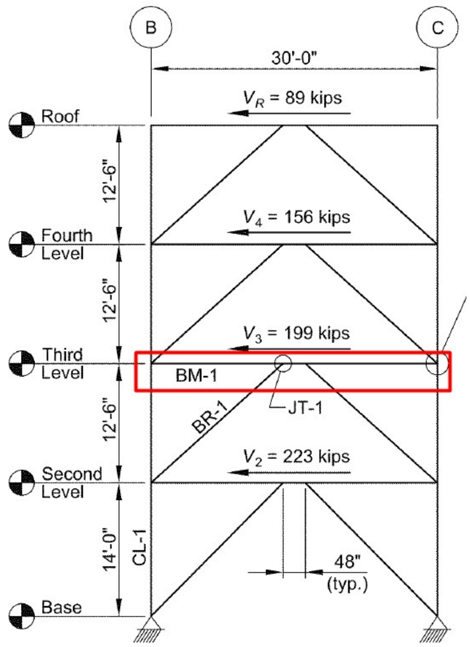
c) Load Case 3 – CBFEM (Deformed shape factor = 1)

Figure 16 – Behavior of link for the variation of design loads in connection – CBFEM

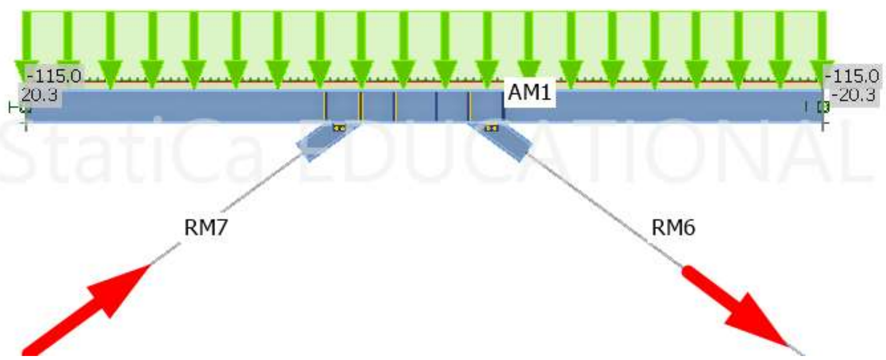
Overall, it can be concluded that the connection will fail at 538kips, which is 12% more than the design load, before a failure is observed, which will be observed in the web of link, such that plastic strains of around 5% are reached.

7. Modelling and Analysis of link member

After evaluating the brace-to-beam connection, the next step in EBF is evaluation of members using IDEA StatiCa Member, which is an engineering software utilizing CBFEM and advanced analysis of shell models to assess the design of members, including their connections, or a substructure, such as EBF. The CBFEM model for brace-to-beam connection and the associated members in the third level of considered eccentrically braced frame (EBF) was prepared in IDEA StatiCa Member V24.1 as shown in Figure 17.



a) Elevation view of EBF (AISC 341, 2016)



b) Member and Connection - CBFEM

Figure 17 – Evaluation of link in EBF

The analyzed member in IDEA StatiCa Member is defined as an investigated member, which is the beam (link and beam outside of link) for this example, upon which floor loads are directly applied and modeled fully with shell elements in CBFEM. The loads on this analyzed member can be applied to the member centerline or directly to the individual plates of the member. The ends of the beam are modelled as fixed support for the performed example. Self-weight is not applied automatically; only the user-defined loads are considered.

Brace members which are the related members in IDEA StatiCa Member are also modeled by shell elements. The ends of related members are supported by user-defined restriction of translation or rotation in an arbitrary direction in the local coordinates of the related member. Internal forces at the end sections of related members are applied as axial forces on related members. The ends of the brace are modelled as pinned support for the performed example.

The following loads are applied:

- Line loads on analyzed members (beam)
- Axial forces in end sections of related members (brace)

The brace-to-beam connection for both brace members was modelled in IDEA StatiCa Connection, using the "Edit Connection" option in IDEA StatiCa Member as shown in Figure 18.

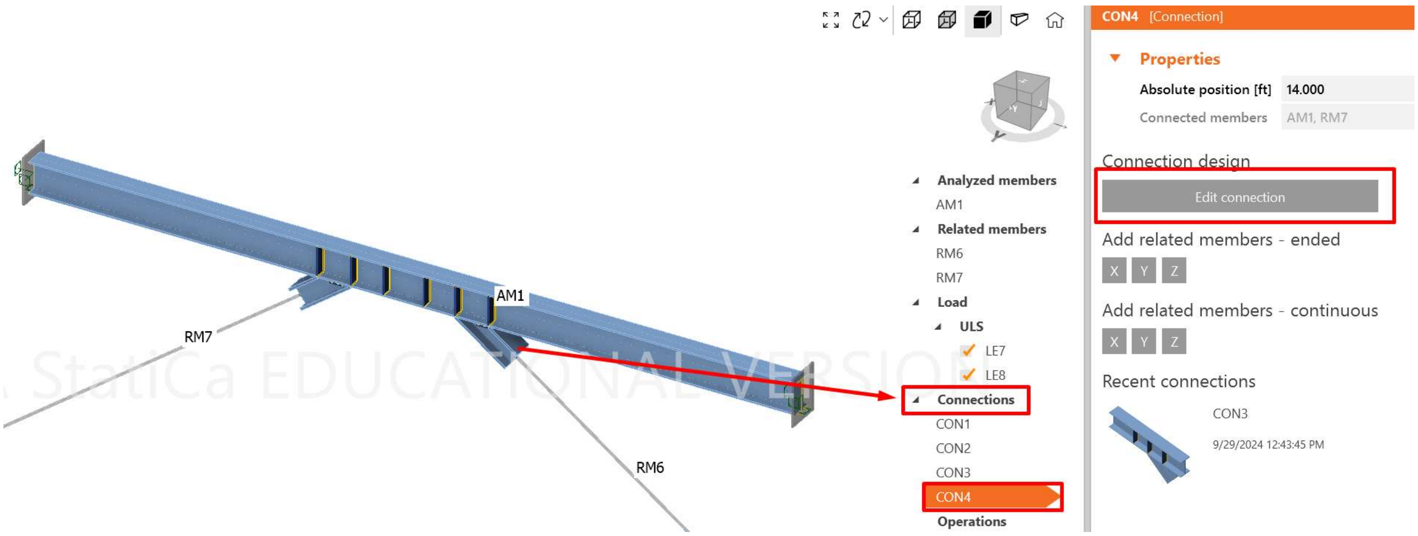


Figure 18 - Modelling of brace-to-beam connection in IDEA StatiCa Member - CBFEM

Standard support is required to be defined at the ends of related members. These supports are defined in the local coordinate system of the member, with support location at the node. The stiffeners in the beam member of EBF are modelled using the manufacturing operations as shown in Figure 19 in IDEA StatiCa Member

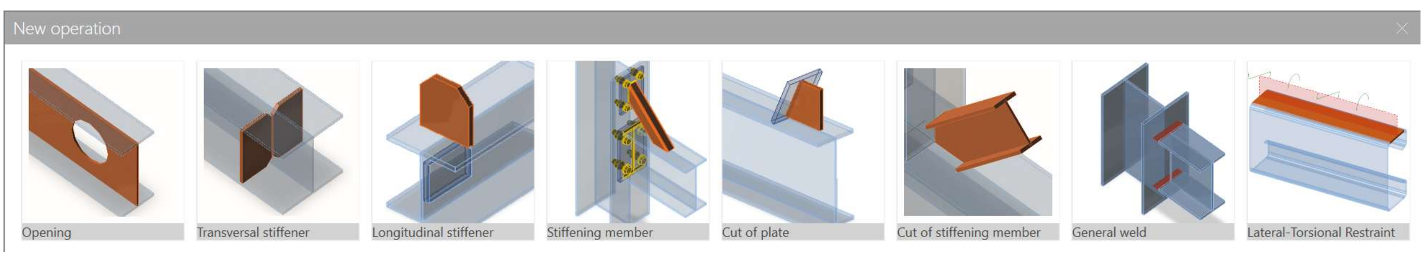


Figure 19 - Manufacturing operations in IDEA StatiCa Member

To consider the capacity design for beam members, their material properties were modified to consider the overstrength factors, since the link is the dissipative item for such conditions as shown in Figure 20.

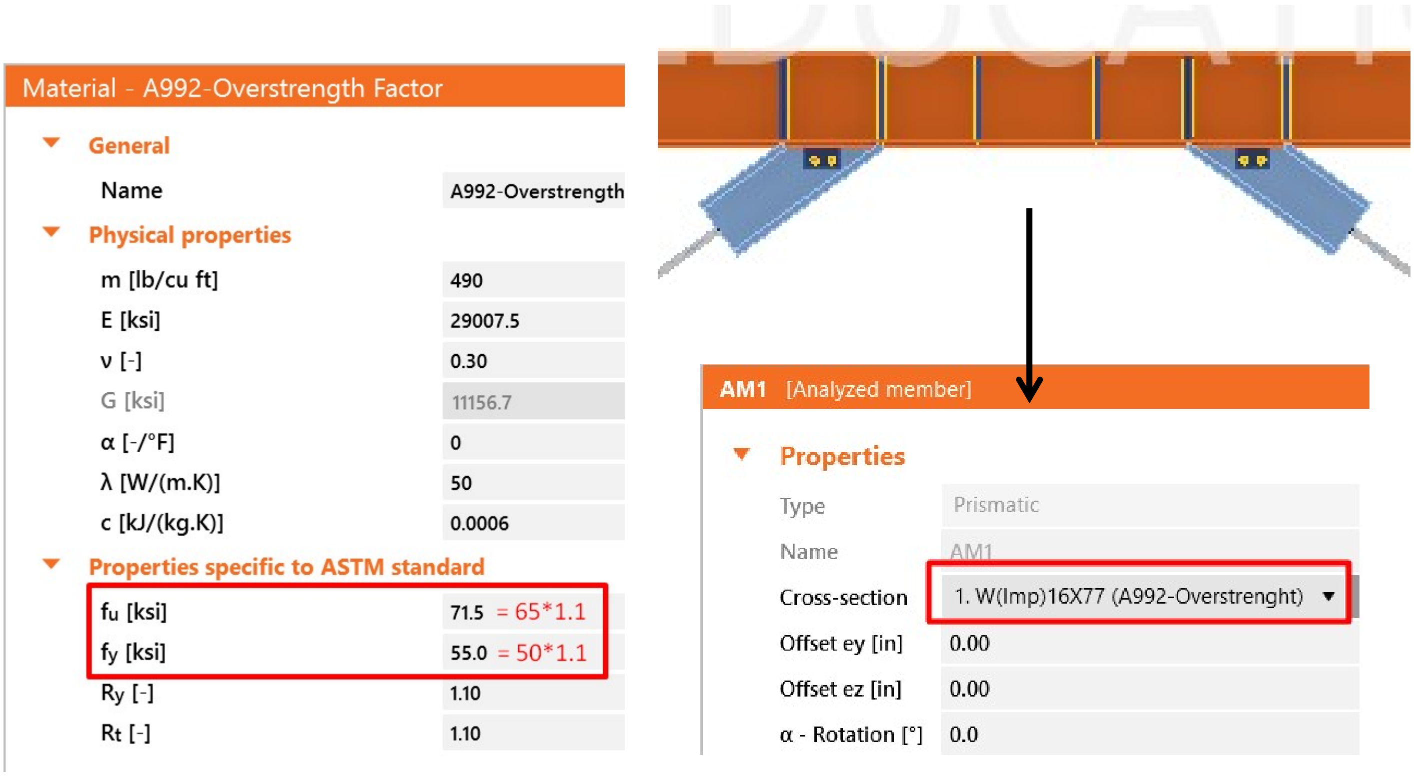


Figure 20 – Material overstrength for beam member in EBF - CBFEM

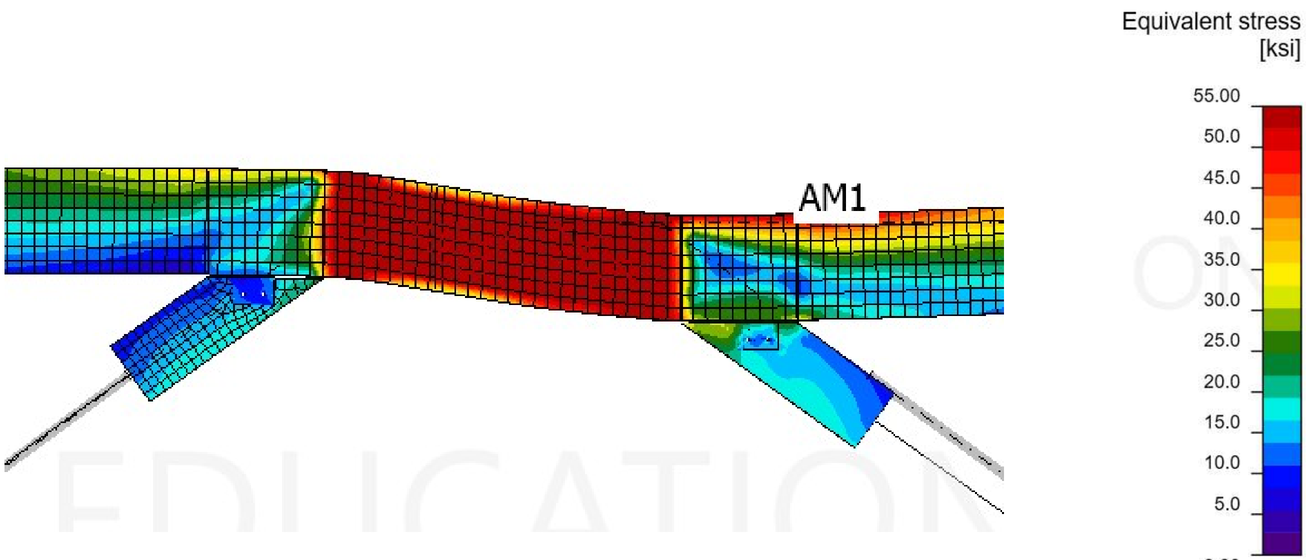
8. Evaluation of link in CBFEM

IDEA StatiCa Member provides three types of analysis, as follows,

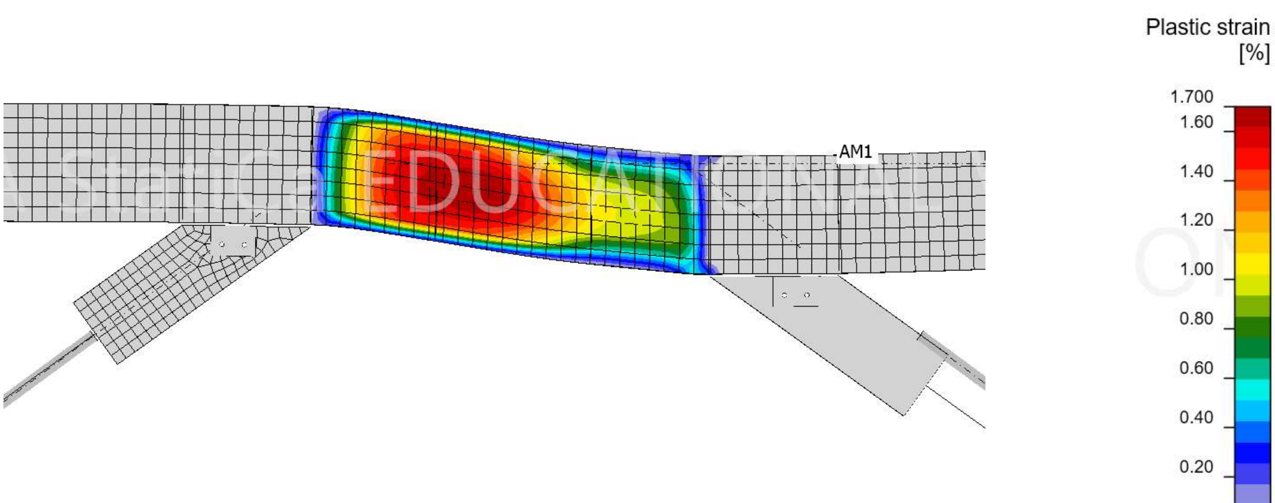
1. MNA- Material Non-linear analysis.
2. LBA – Linear Buckling Analysis (stability).
3. GMNIA – Geometrically and Materially Non-linear analysis with imperfections.

For MNA, a strain check of 5% is applied. While LBA tells us about the shape of stability collapse and further advises on how imperfections should be defined. Afterwards, GMNIA is used, and the strain check of 5% is again applied for the attainment of maximum load towards the end of convergence.

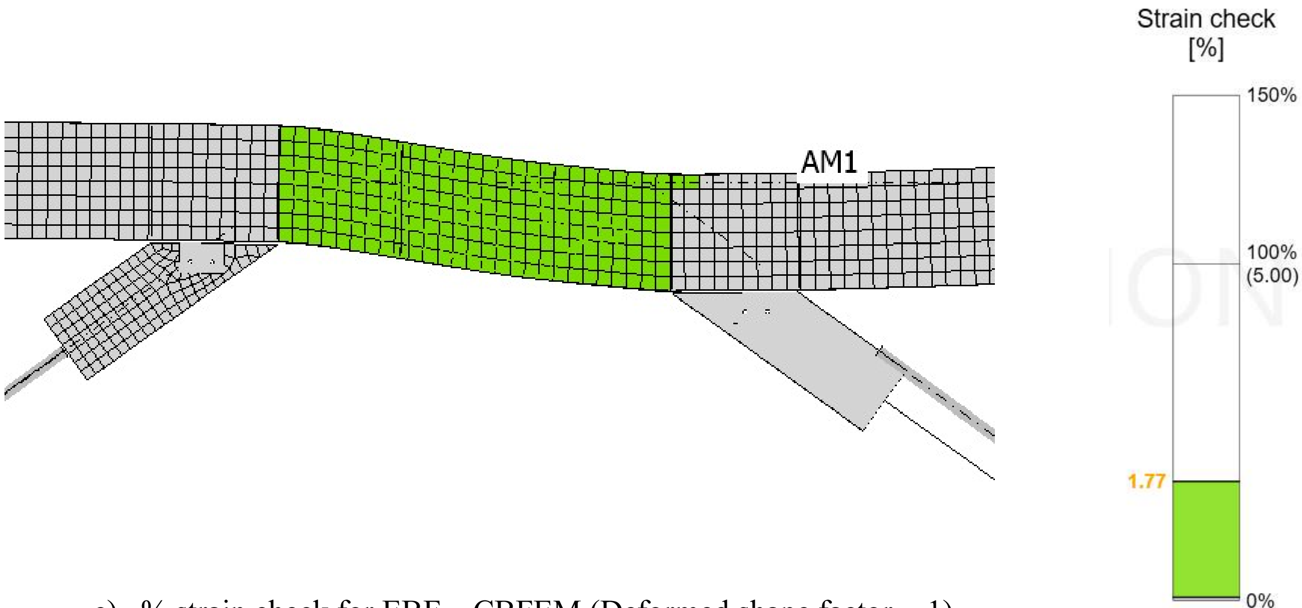
Based on the analysis results for the MNA, the member is found to be safe, such that the analysis reached 100%, The plastic strain in plates is 1.7% < 5% and are observed in the link as shown in Figure 21.



a) von Mises stresses in link– CBFEM (Deformed shape factor = 1)



b) Plastic strains in link– CBFEM (Deformed shape factor = 1)



c) % strain check for EBF – CBFEM (Deformed shape factor = 1)

Figure 21 – MNA for evaluation of link for design loads - CBFEM

Next, LBA was performed, such that buckling modes are calculated by Lanczos algorithm in software. The structure is considered perfect without any geometrical or material imperfections, and the material is elastic in this analysis type. Linear buckling analysis provides factor α_{cr} – minimum amplifier for design loads to reach the elastic critical resistance of the structural component. The factor determines the load when Euler's critical buckling load is reached.

For mode shape 1 as shown in Figure 22, which is a critical buckling mode, obtained buckling factor is 1.85 and observed in the brace member. This buckling type can be classified as global buckling.

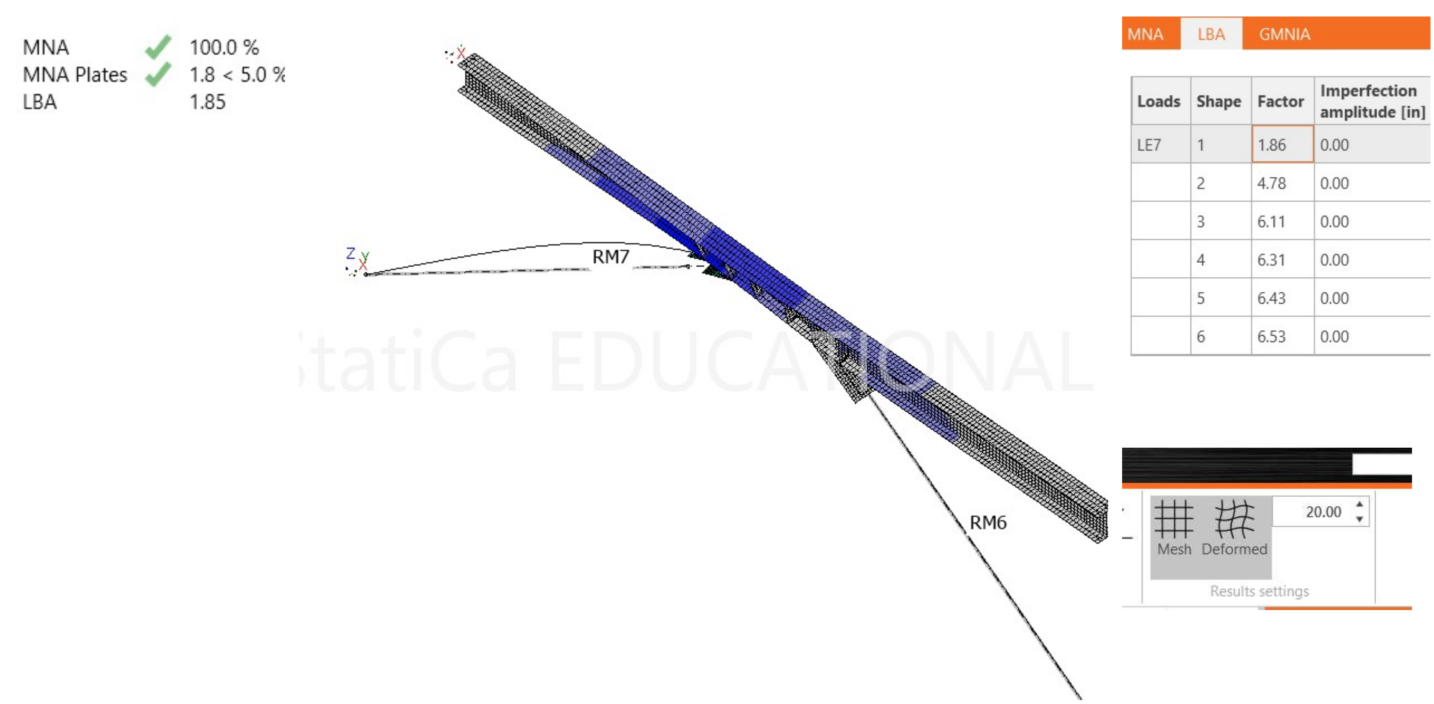


Figure 22 - Buckling Shape for Mode 1 in member (Deformed shape factor = 20) – CBFEM

While for mode shape 2 as shown in Figure 23, the buckling factor was 4.78 > 3 and was observed in beam outside of link.

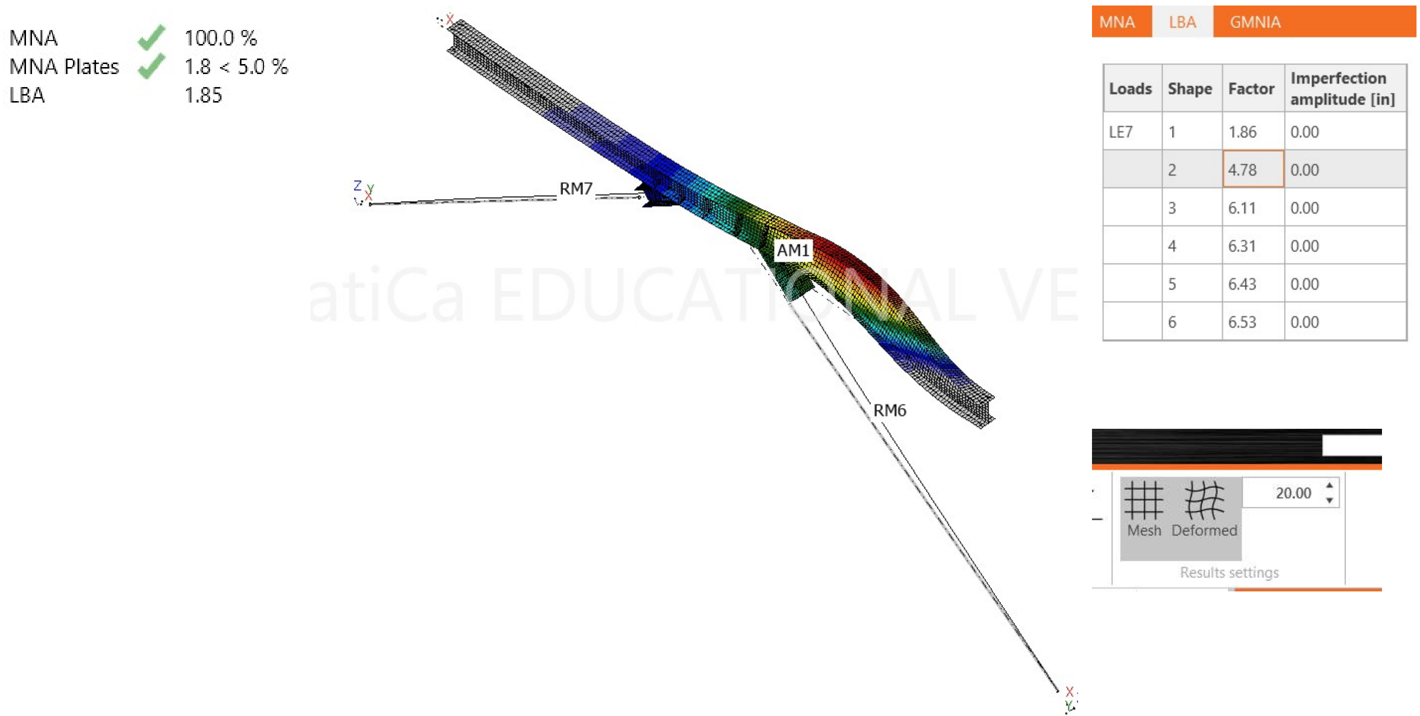
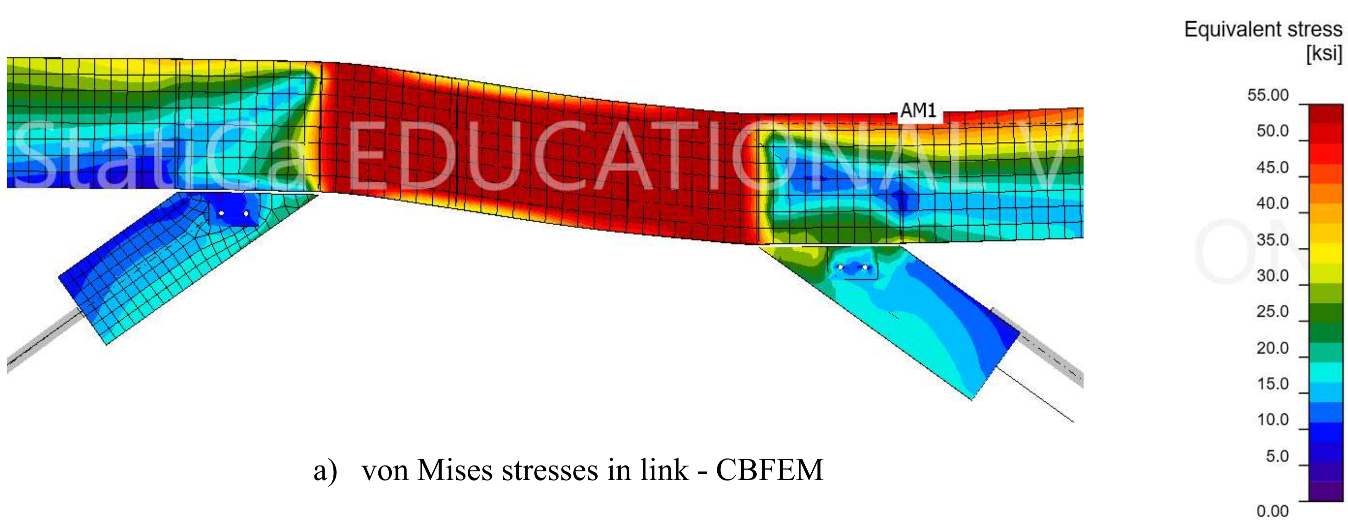


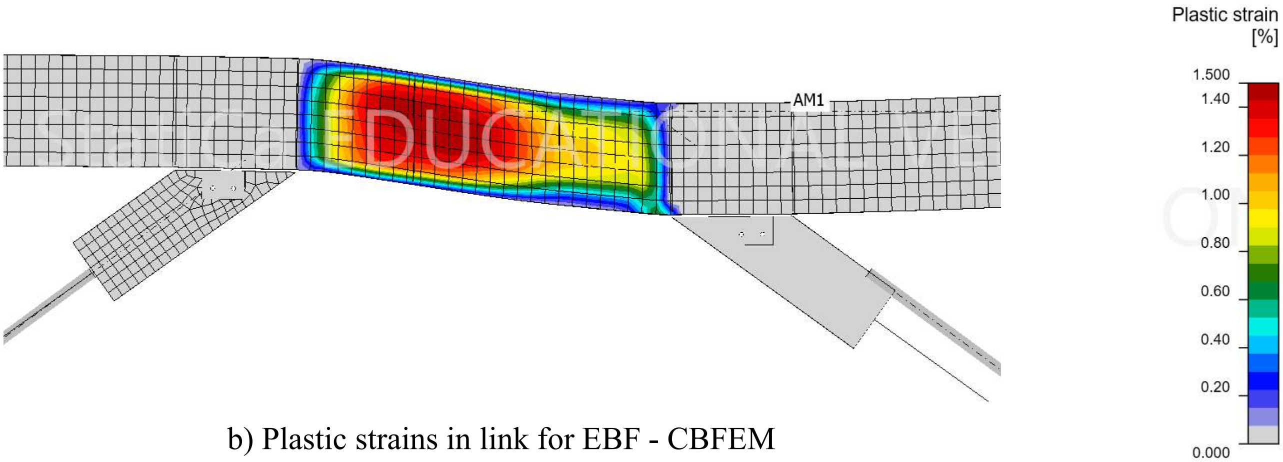
Figure 23 - Buckling Shape for Mode 2 in member (Deformed shape factor = 20) - CBFEM

Since the critical buckling factor is less than the recommended buckling factor of 15 for a member, GMNIA is performed to evaluate member stability.

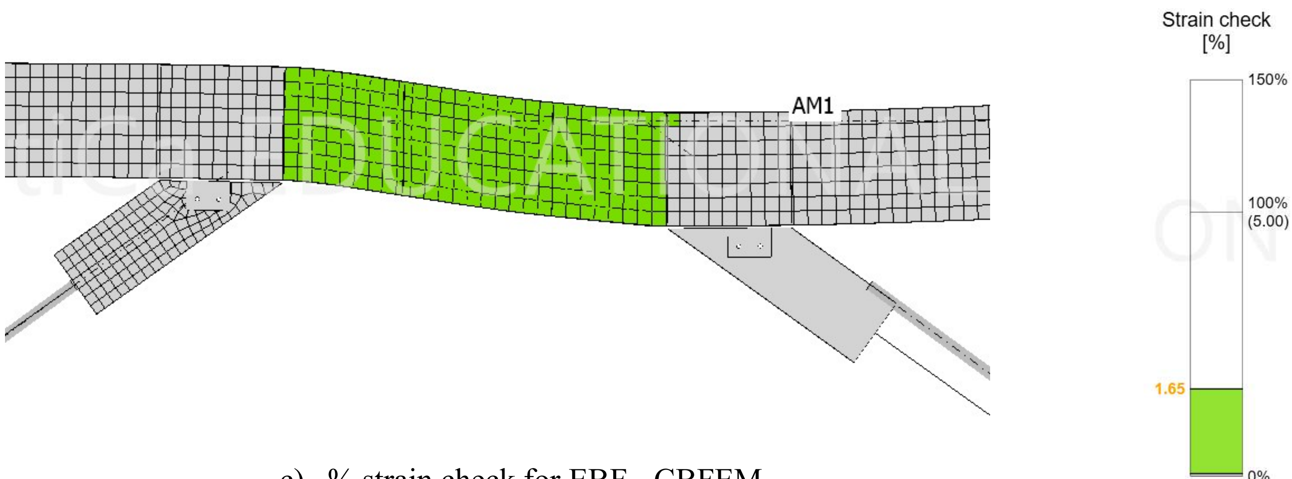
In GMNIA, all the imperfections (varying thickness of plates, out-of-straightness, residual stresses, non-homogeneities in material, misalignment of supports etc.) are substituted by equivalent geometrical imperfections and can be set using buckling mode shapes calculated by LBA. The member is found to be safe for the GMNIA, such that the analysis is running 100% and plastic strain in plates is 1.6% < 5% as shown in Figure 24.



a) von Mises stresses in link - CBFEM



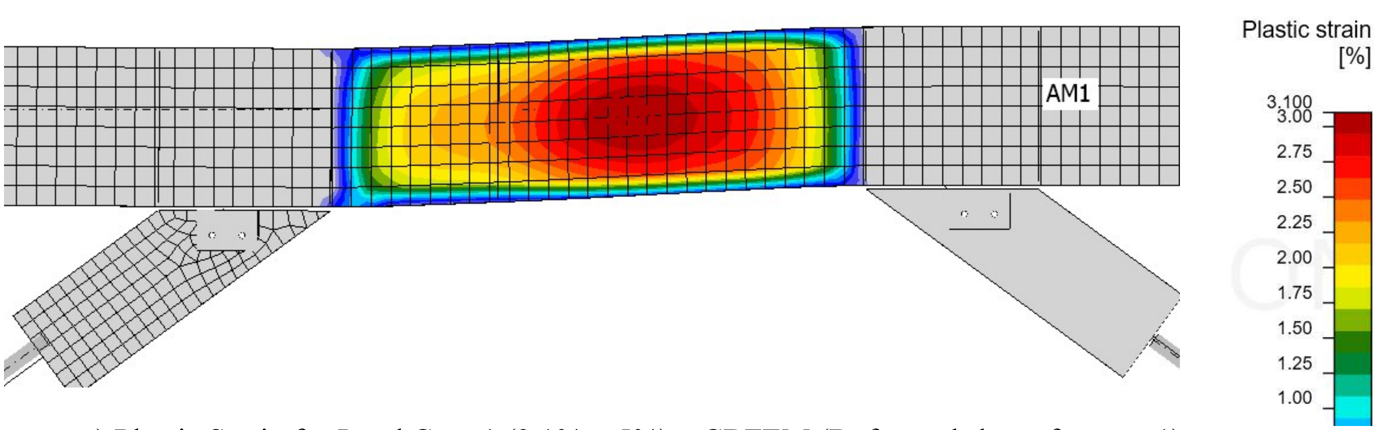
b) Plastic strains in link for EBF - CBFEM



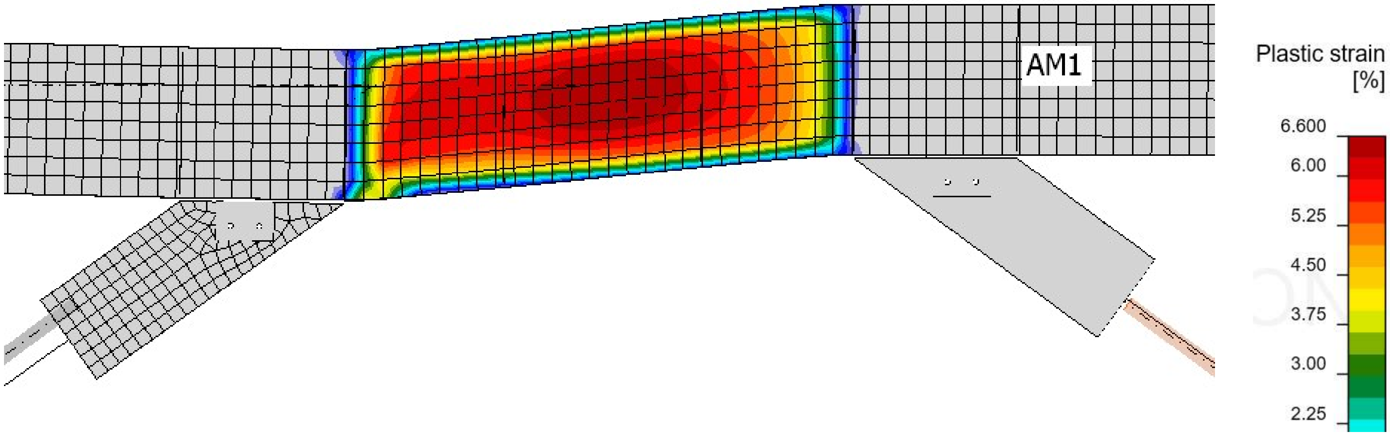
c) % strain check for EBF - CBFEM

Figure 24 - GMNIA for evaluation of link for design loads - CBFEM

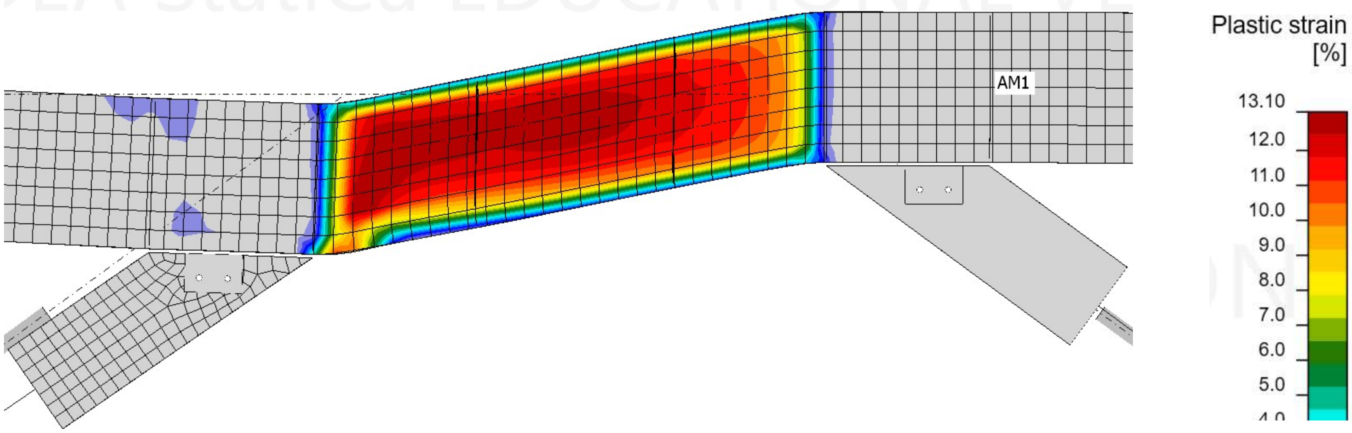
To understand the link behavior in StatiCa member for the variation in design loads, the load cases as obtained from Table 1 as considered. Based on the results obtained, the link is found to exhibit excellent energy dissipation, such that all the plastic strains are concentrated in the link only for the action of higher loads as shown in Figure 25.



a) Plastic Strain for Load Case 1 (3.1% < 5%) – CBFEM (Deformed shape factor = 1)



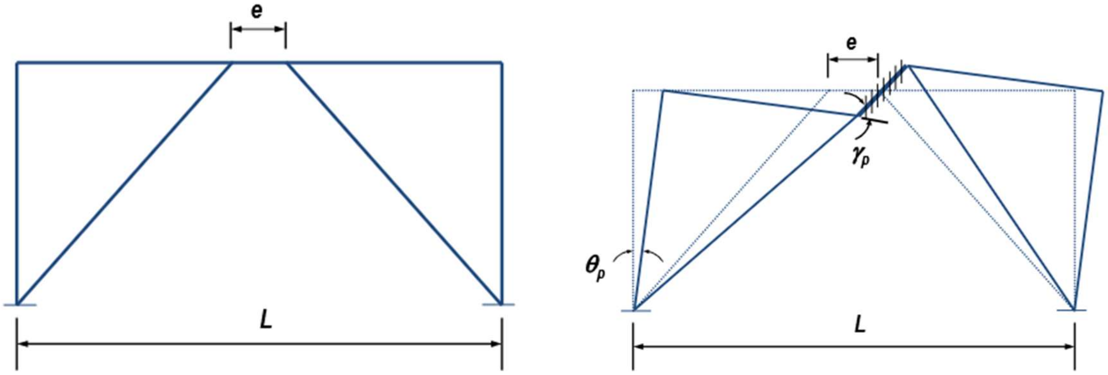
b) Plastic Strain for Load Case 2 (6.6% > 5%) - CBFEM (Deformed shape factor = 1)



c) Plastic Strain for Load Case 3 (13.1% > 5%) - CBFEM (Deformed shape factor = 1)

Figure 25- Behavior of link for the variation in design loads in StatiCa Member

The link rotation angle (γ_p) as shown in Figure 26, is defined as the inelastic angle between the link and beam outside of the link when the story drift is equal to the design story drift (Δ), as mentioned in Section F3.4a of AISC341-16.



$$\gamma_p = \frac{L}{e}\theta_p$$

Figure 26- Link rotation angle in EBF (AISC 341, 2016)

AISC Seismic Provisions Section F3.4a specifies a maximum link rotation angle which is based on the expected behavior of the link and as follows,

a) For link of length (e) equal to ($1.6M_p/V_p$) or less; the link rotation angle shall not exceed 0.08radians.
b) For link of length (e) equal to ($2.6M_p/V_p$) or greater, the link rotation angle shall not exceed 0.02radians.
c) For link of length (e) between ($2.6M_p/V_p$) and ($1.6M_p/V_p$), the link rotation angle shall be based on the linear interpolation.

where, M_p is plastic bending moment of the link in kip-ft, and V_p is plastic shear strength of the link in kips.

In CBFEM, the link rotation angle γ_{p_CBFEM} can be calculated by using simple trigonometry for the results of deformation in vertical direction Uz (in) as shown in Figure 23,

$$\gamma_{p\_CBFEM} = \tan^{-1} (\Delta_{Uz} / e)$$

where, e is the link length and Δ_{Uz} is the difference in the vertical deflection of the link as obtained from CBFEM results and shown in Figure 27.

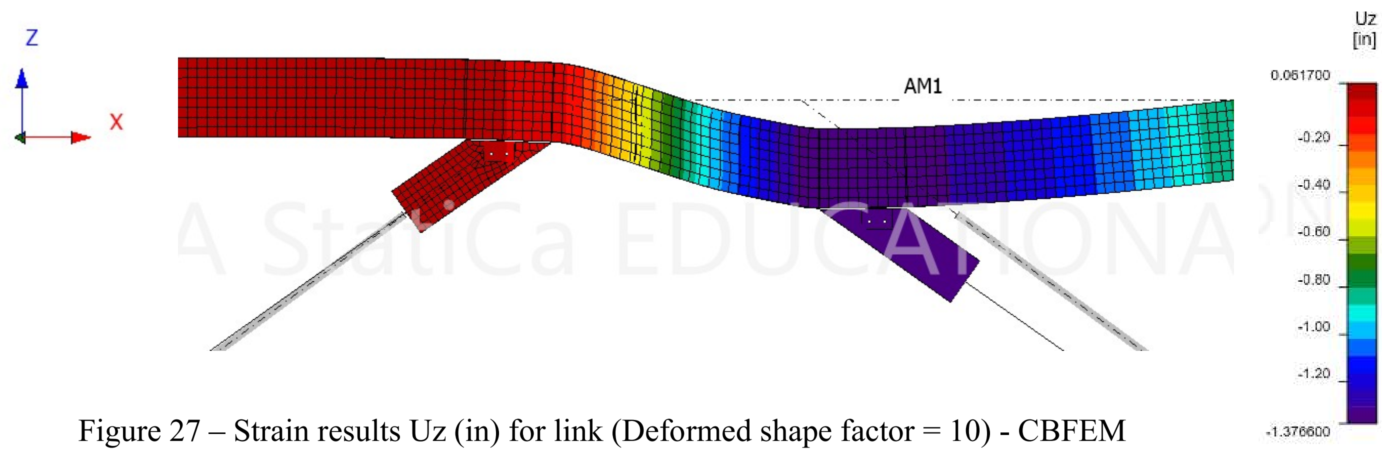


Figure 27 – Strain results Uz (in) for link (Deformed shape factor = 10) - CBFEM

As per the CBFEM results, the obtained link rotation angle of 0.04radians < 0.08radians (more details in https://www.dropbox.com/scl/fi/rdca1t0p30jcyn34y1yry/EBF-Rotation-Link-PDF.pdf?rlkey=wpqeov51e19i8u7hzoxgath8h&st=911tkeh5&dl=0), is observed to satisfy the AISC requirement for short link wherein shear yielding is predominant.

A study was performed, wherein the intermediate link and long link are evaluated in CBFEM, and the obtained rotation angle is compared with that from AISC equations. The results are as tabulated in Table 3,

Sr No.	Link Details			Link Rotation Angle (γ_p)		
	Type	Behavior governed by	Length (e)	Shall not exceed	γ_{p_AISC}	γ_{p_CBFEM}
1	Short	Shear yielding	48in	0.08 radians	0.03 radians	0.04 radians
2	Intermediate	Combined shear and flexural yielding	80in	0.04 radians (from linear interpolation)	0.02 radians	0.02 radians

3	Long	Flexural yielding	98in	0.02 radians	0.01 radians	0.01 radians

Table 3 – Evaluation of link rotation angle – CBFEM

Overall, it can be concluded that the CBFEM in IDEA StatiCa Member yields similar results for link rotation angle of link in an EBF.

9. Flowcharts

The pdf files of flowcharts are in the Flowchart folder of EBF file.

https://www.dropbox.com/scl/fo/nydkocorc0xzyndq384mo/ADsRVCKN1nLAaiU-kdzoh48?rlkey=cbo7mtycyzkd7x1dvqt7h6z9t&st=7nv79fkd&dl=0

10. Summary

The following summarizes, loads, detailing of connections, why use IDEA StatiCa connection and member, and results from the analysis and design of EBF.

Loads used in the analysis:

- The link beam is designed for the seismic loads calculated per ASCE 7.
- The members outside the link beam are designed based on the link capacity in axial, shear, and moment considering R_y and strain hardening factors.
- The connections are designed for the expected forces based on capacity design.

Detailing of connections:

- As EBF are expected to withstand significant inelastic deformation in the link member when subjected to the force resulting from the motions of the design based earthquake (DBE), AISC341-16 has requirements for detailing of link members which includes the spacing and sizing requirements for the stiffeners plates for effective energy dissipation, so as to avoid the failure of the connection and member as a whole.
- Detailing requirements for the link beam need to be investigated, modeled, and checked by the user according to AISC 341-16.

Design of EBF using IDEA StatiCa member and connection:

- **Connections**: Different types of seismic brace-to-beam/column connections are applicable in EBFs, therefore, CBFEM can be used to check the connection for design load and also for several limit states as applicable.
- **Member**: analysis of the EBF members/frame can be performed using CBFEM to understand the behavior for seismic design load. The link beam is designed for seismic forces obtained from ASCE 7. IDEASTATICA Member includes MNA-Material Non-linear Analysis, LBA-Linear Buckling Analysis, and GMNIA-Geometrically and Materially Non-linear analysis with imperfections, which are required to be performed for the analysis of the link beam.

Results and design:

- CBFEM using IDEA StatiCa provides realistic behavior of connection and member, results differ since it is a finite element tool but accurate enough for connection check and member check. See recommendations for improvement above. The difference in results depend on the connection configuration, the verification performed that EBF elements can be analyzed using IDEASTATICA member and connection predicted the behavior with accurate results.

11. Conclusion

Eccentrically Braced Frames (EBFs) are amongst the important braced frame systems that utilize a link beam created by the eccentric connection of the diagonal braces. This system provides excellent energy dissipation by the inelastic deformation of the link. The brace-to-link connection from AISC Seismic Manual was observed to be safe also based on the analysis results from CBFEM. Wherein, the link demonstrates excellent energy dissipation for increased loads in the members, when a capacity design with "link" as dissipative item is performed in CBFEM.

Evaluation of link for CBFEM in IDEA StatiCa Member was performed with three analyses – Material non-linear analysis (MNA), linear buckling analysis (LBA) and geometrically and materially non-linear analysis with imperfections (GMNIA). As per MNA, all plastic strains are observed in link such that 1.7%<5%. As the critical buckling factor from LBA was less than recommended buckling factor of 15 for members, GMNIA was performed to evaluate member stability. The link is observed to be stable and provide excellent energy dissipation, with all plastic strain concentrated in the web of link as observed from results of GMNIA.

The link rotation angle (γ_p) which is defined as the inelastic angle between the link and beam outside of the link when the story drift is equal to the design story drift (Δ), was evaluated in CBFEM for three types of link– Short, Intermediate and Long as per AISC341-16. The cause of collapse at short link is shear yielding on the web, while at intermediate link is the combination of shear and bending yielding, and for long link is bending yielding, while the beams, columns and brace members are expected to remain elastic. The link rotation angle value from CBFEM yields similar results to that from AISC.

12. Recommendation for Improvements

The following are recommendations for improvements based on the process used to check and design EBF members and connections.

a. IDEA Connection and EBF
   i. Joint design resistance for seismic connection
   ii. Loads in equilibrium for capacity design analysis.

iii. Detailing check for connection in CBFEM as per AISC 341

b. IDEA Member wrt EBF (mentioned points are not possible with current version)
   i. "Capacity design" analysis in StatiCa member.
   ii. "Member design resistance" for members in StatiCa member.
   iii. To perform a check for link rotation angle as per AISC, without needing to check it manually.
   iv. Detailing check for member in CBFEM as per AISC 341.

13. Future Scope for work (Six items as mentioned below)

a. GMNIA with Imperfection for IDEA StatiCa member as per AISC

Currently there exist work for GMNIA with imperfection for IDEA StatiCa member as per Eurocode only, and the same for AISC can be a part of future scope.

- https://www.ideastatica.com/support-center/geometrically-and-materially-nonlinear-analysis-with-imperfections-gmnia-of-beams-in-bending
- https://www.ideastatica.com/support-center/geometrically-and-materially-nonlinear-analysis-with-imperfections-gmnia-of-beams-in-bending
- https://www.ideastatica.com/support-center/how-to-input-imperfections-in-member

b. Evaluation of brace-to-beam/column connection (Example 5.4.7 in AISC 341-16) in EBF using stress-strain analysis in CBFEM

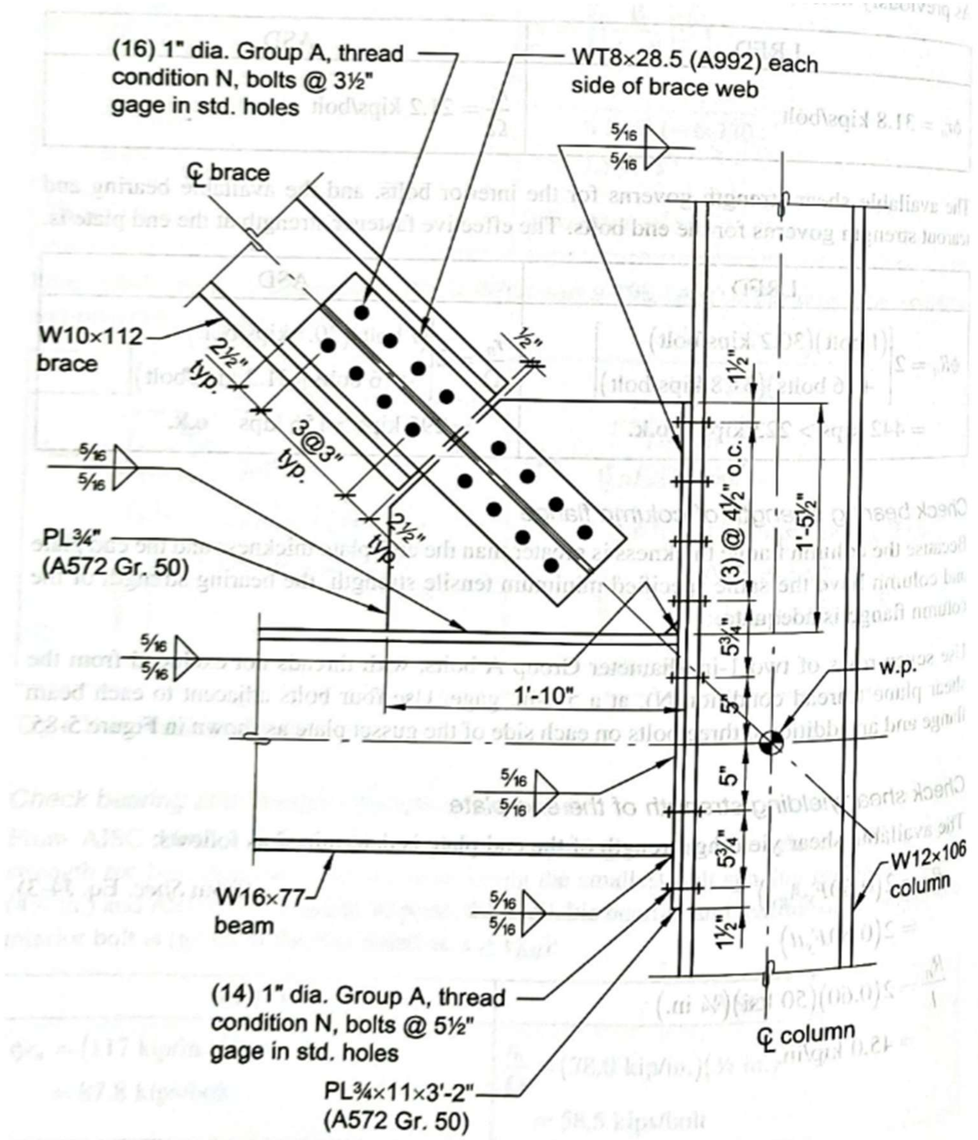


Fig. 5-85. Connection designed in Example 5.4.7.

c. Evaluation of EBF connections and members with replaceable link members.

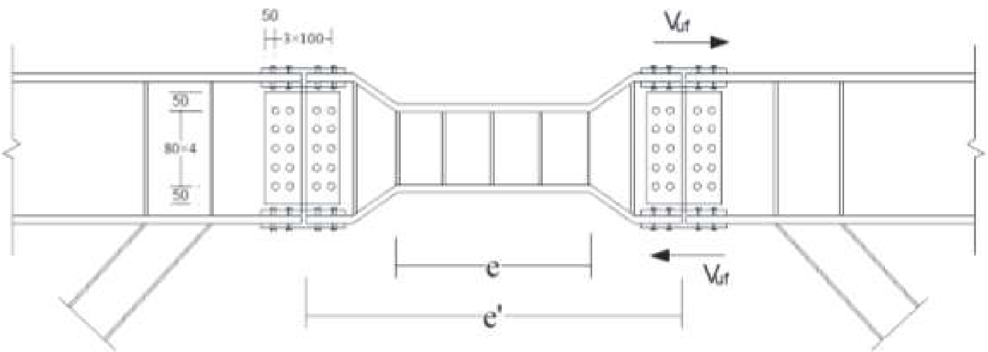


Figure 15. Bolt connection design of link.

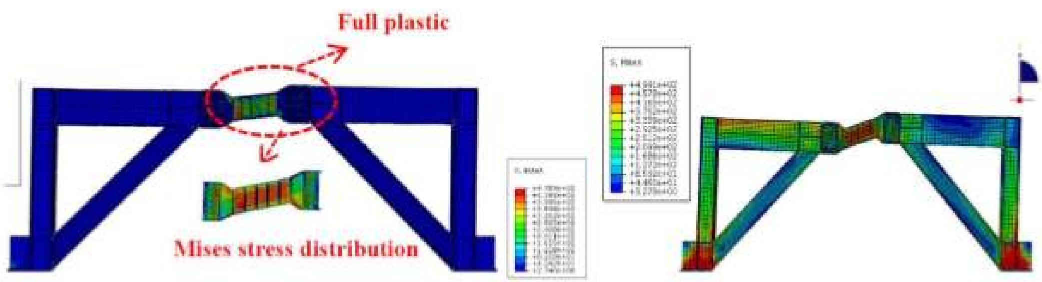


Figure 20. Failure mode of the base mode.

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d. Evaluation of CBFEM for various Link-to-Column Connections in EBF

Refer to https://www.iitk.ac.in/nicee/wcee/article/13_275.pdf for more information.

i. Secondary Pre-Northridge Connections

4.1. Secondary Pre-Northridge Connections

The configurations of these types of connections are as the same connections as were tested by Okazaki and Engelhardt. The analytical results obtained from finite element method shows that the beginning of fracture at the end of weld access hole is happened at similar plastic rotation value indicated in the experimental results. A comparison between analytical and experimental results is shown in figure.3-b. Both results are also compared with code provisions in this figure.

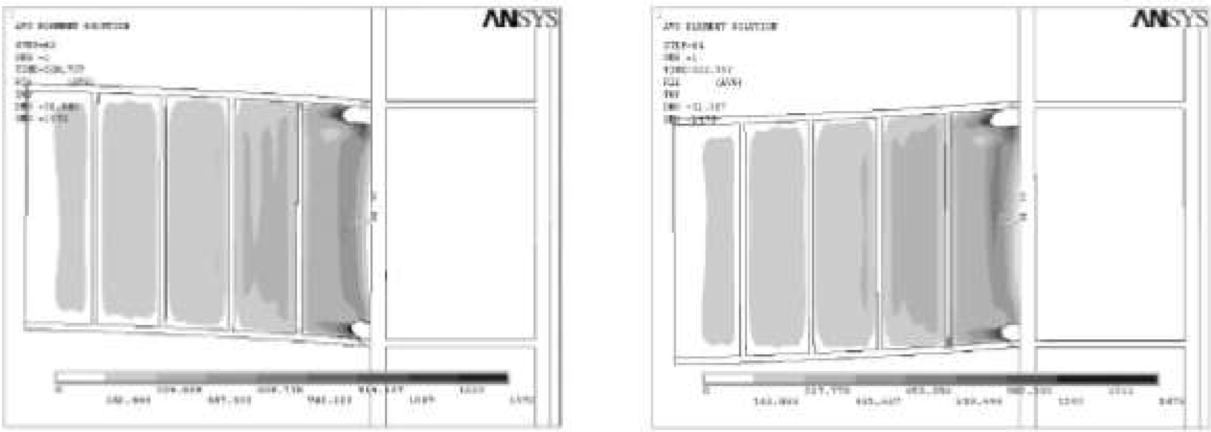


Figure 2. Prediction of location of fracture in short link with PN connection ($\gamma_p = 0.05^{rad}$)

ii. Free Flange Connections

4.2. Free Flange Connections

These types of connections are modeled according to the experimental details tested by Okazaki and Engelhardt. The results from finite element analysis show that the fracture in short link happens at the web of the link, around shear tab (similar to the case of test results) at plastic rotation equal to 0.07 rad (Figure 5). It was also shown that the fracture in the long link happened at the shear tab/link web groove weld similar to the location in experimental results. The plastic rotation was equal to 0.02 rad (Figure 6). A comparison between analytical and experimental investigation is shown in figure 3-b.

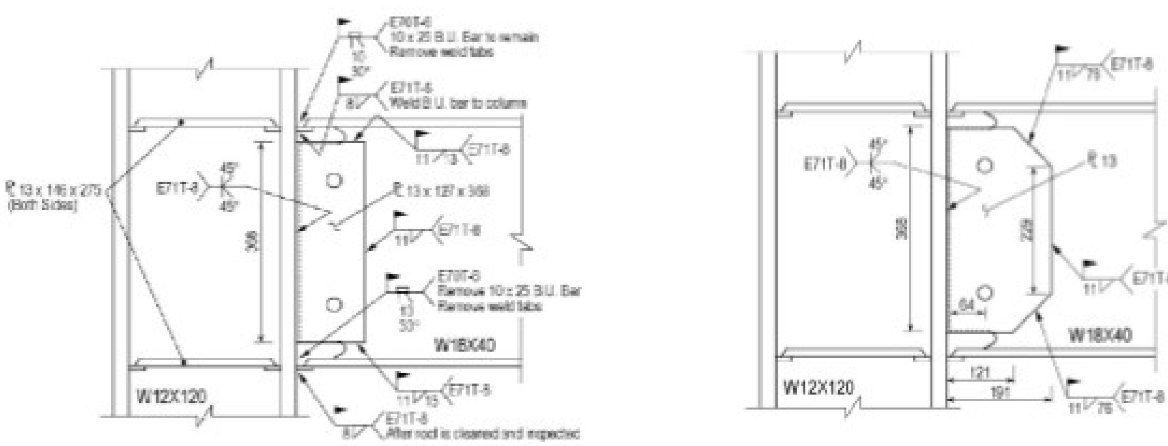


Figure 4. Connection details for short and long link

iii. Connections with Reduced Beam Section (RBS)

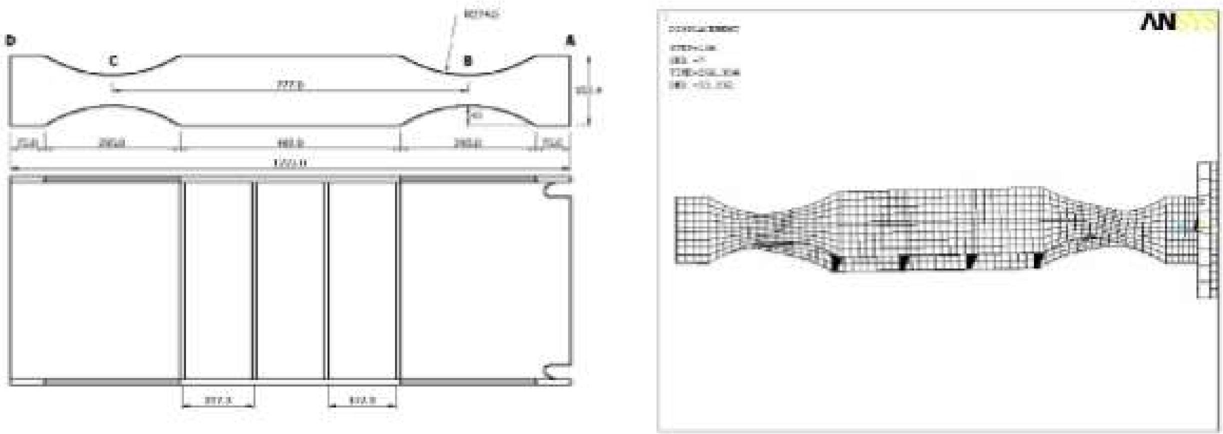


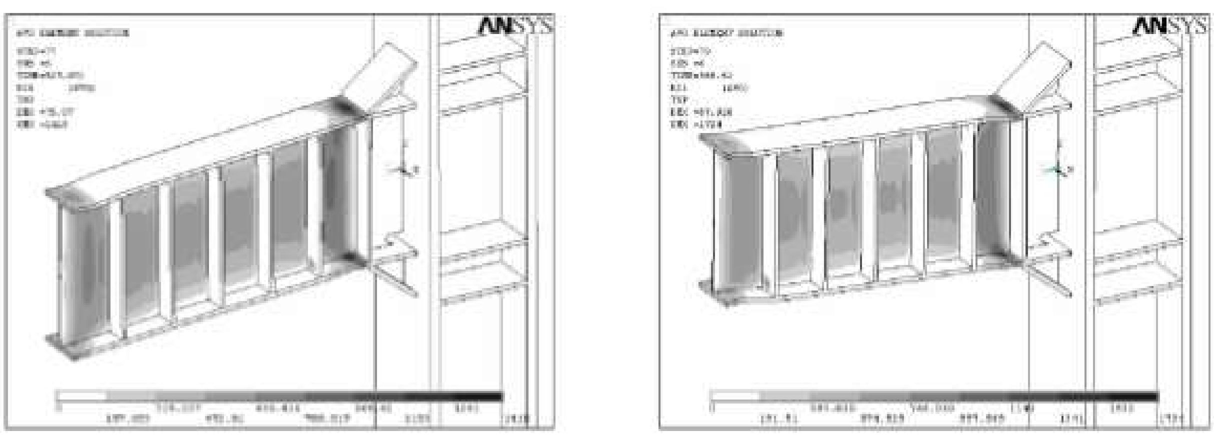
Figure 8. a. reduced section properties for intermediate link

iv. Welded top and bottom haunch connections

4.4. Welded top and bottom haunch connections

These types of connections were designed based on procedure of welded bottom haunch connections which were suggested by Yu and Uang. Their method was based on design of top and bottom haunch (double haunch).

In this type of connections, the failure mode concentrated at the toe of haunch. This sort of failure is the main failure mode that can be happened in the links (if the buckling mode is prevented). Although the short link could not achieve the value suggested in code provision, but in other links (intermediate and long links), the ultimate capacity of the connections is very close to the code provisions value.



Currently, there are no prequalified connections for link-to-columns for EBF and this stands as future scope for overall research.

e. Providing brace as BRB in EBF and providing beam splices

https://www.researchgate.net/publication/268412873_Using_Buckling-Restrained_Braces_in_Eccentric_Configurations

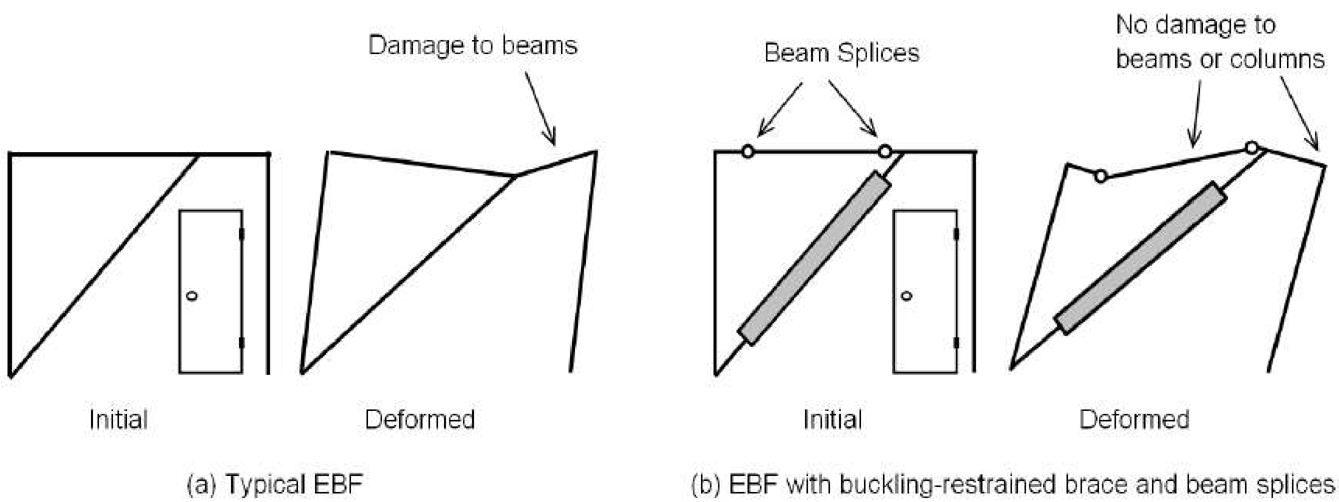


Fig. 1-7 Typical EBF and proposed EBF with buckling-restrained brace and beam splices

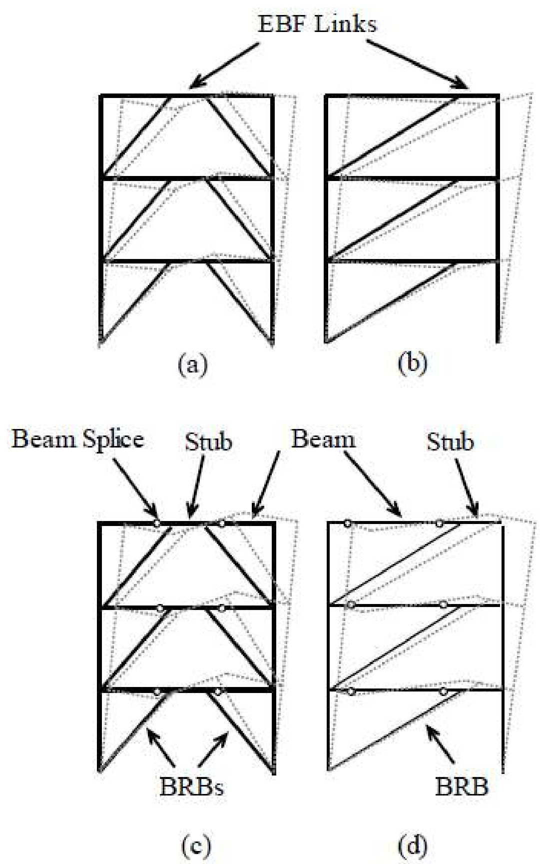


Fig. 3-1 Initial and deformed configurations for (a-b) EBFs and (c-d) BRBF-Es

f. Evaluation of vertical shear link connection in EBF

Refer to https://www.sciencedirect.com/science/article/abs/pii/S2352710216303114 for more information.

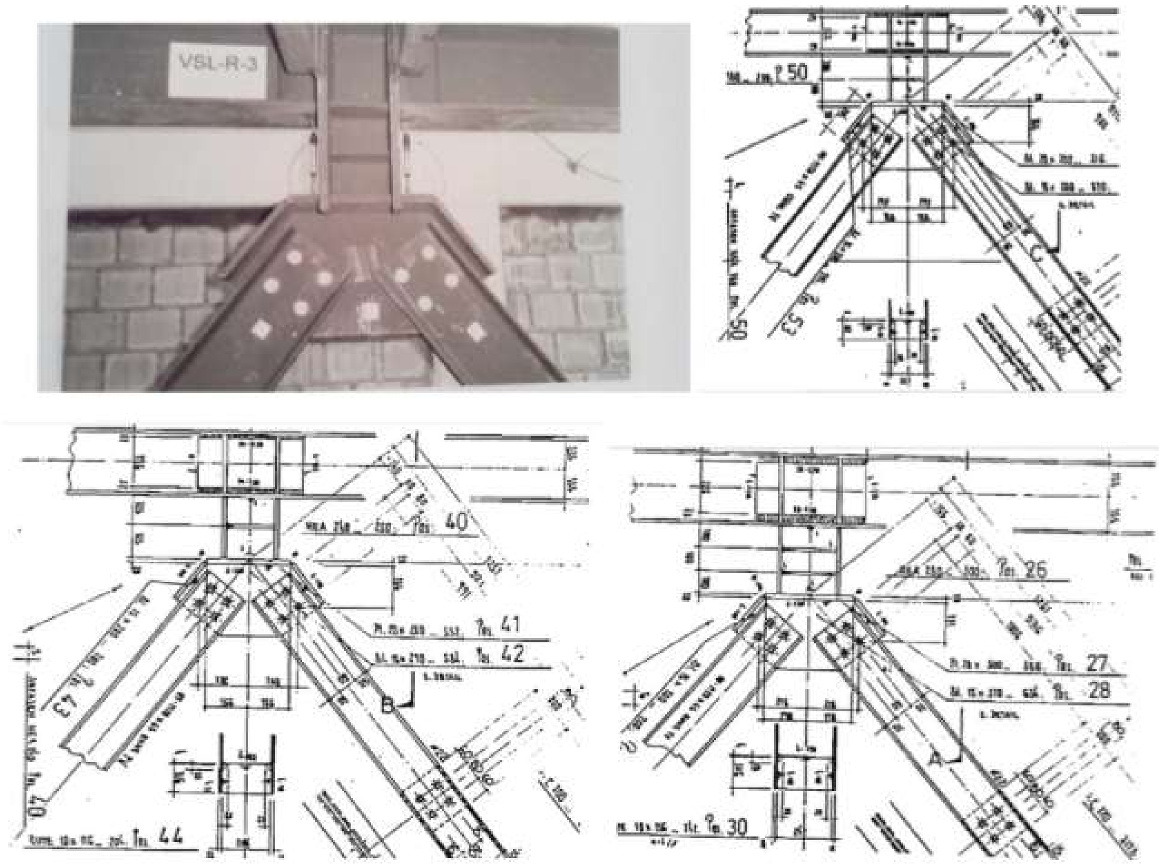


Fig. 6. Connection details of the specimens.

14. References

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