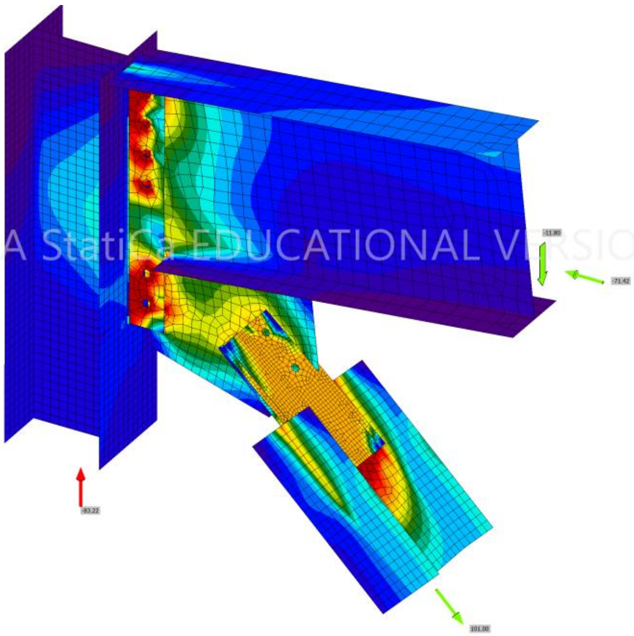
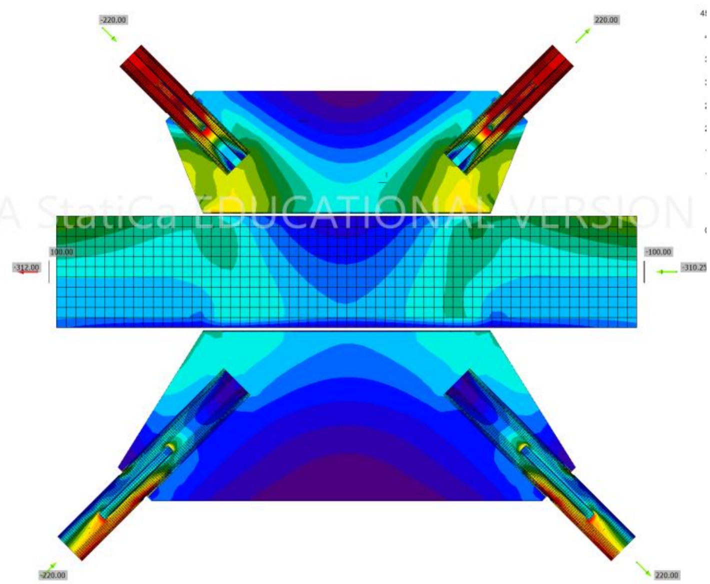
Connections in Ordinary Concentrically Braced Frame (OCBF) System





1.1 Introduction

Braced frames are a common type of seismic lateral force resisting system used in steel structures. The two major types of Concentric Braced Frames (CBFs) are Ordinary Concentrically Braced Frame (OCBF) and Special Concentrically Braced Frame (SCBF), while the Buckling-Restrained Braced Frame (BRBF) is a special type of concentric braced frame. As illustrated in Figure 1, the braces within a frame are laid out such that the centerlines of the connecting elements (beams, braces, and columns) intersect each other at a common point of connection known as work point.

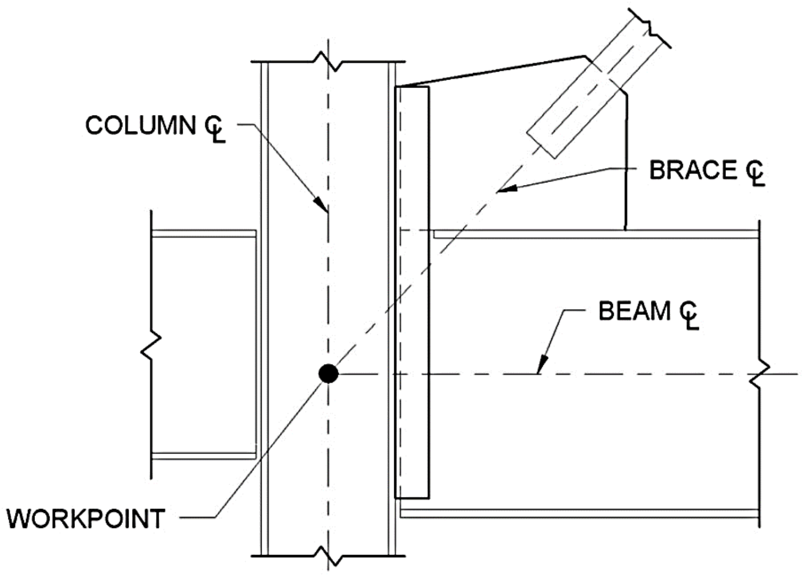


Figure 1- Intersection of Member Centerlines (Grusenmeyer, 2012)

Several configurations of braces are possible in concentric braced frames (CBF). Most commonly used layouts are V-, inverted-V-, X- and K-frames as shown in Figure 2. Each layout has its own advantages and disadvantages for the design, structural performance, fabrication, and construction. However, they all function in a similar pattern when subjected to action of design loads.

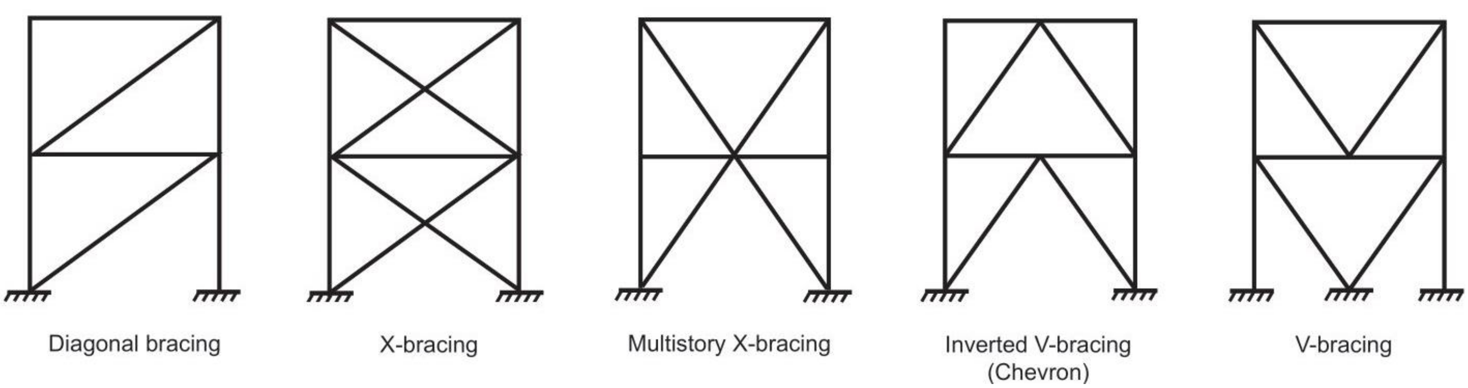


Figure 2 - Commonly used brace layouts in concentrically braced frames (Sabelli et al., 2013)

1.2 Overview of OCBF

Concentrically braced frame systems tend to be more economical than Moment-Resisting Frames (MRF) and Eccentrically Braced Frames (EBF) in terms of material, fabrication, and erection costs. However, they have reduced flexibility in floor-plan layout, space planning and electrical and mechanical routing because of the presence of braces. In certain circumstances, however, braced frames are exposed and featured in the architecture of the building, as shown in Figure 3.

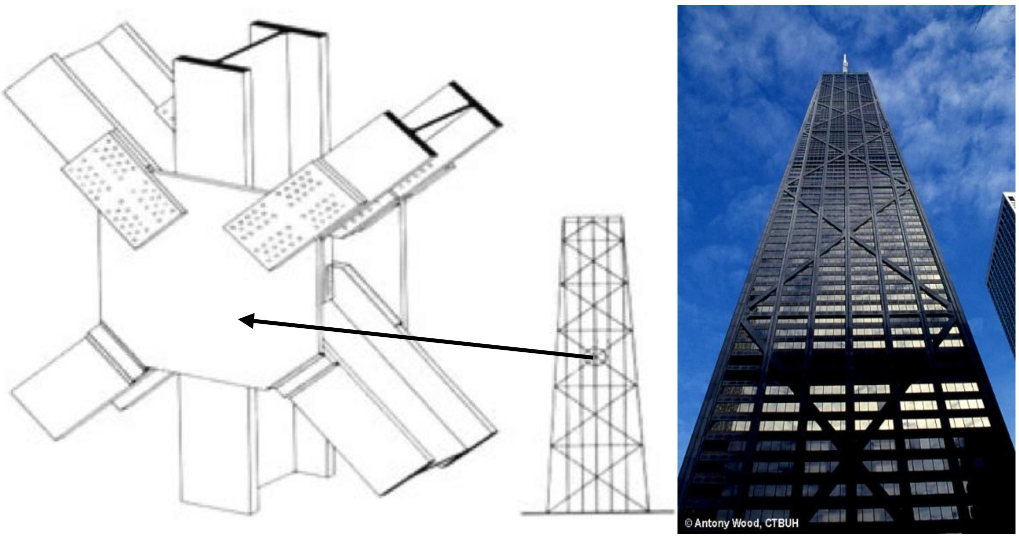


Figure 3 - John Hancock Center, Chicago (Vandenbempt, 2018)

Braced frames are typically located in walls that stack vertically between floor levels. In the typical office building, these walls generally occur in the core area around the stairs and elevator shafts, central restrooms, and mechanical and electrical rooms. This generally allows for greater flexibility in the placement and configuration of exterior windows and cladding. Depending on the location in the plan and the size of the core area of the building, the torsional resistance offered by the braced frames may become a controlling design parameter. Interstory drifts at the perimeter of the building must be considered carefully with this type of layout because rotational displacements of the floor diaphragm may result in perimeter displacement or drifts that impose forces on the cladding system and other non-structural components.

Multi-tiered brace frames (MTBF) are those frames in which brace axial forces are transmitted to other braces, either directly or through a beam acting as an axial strut, at a location lacking out-of-plane support for stability as shown in Figure 4. In typical frames, such out-of-plan stability is provided by beams or floor diaphragm engaging the orthogonal lateral system. The lack of out-of-plane support in MTBF requires the columns to have significantly higher out-of-plane flexural strength and stiffness, which is then reflected in the unbraced length. Additionally, if the deformation of the individual tiers is not uniform, the columns will experience in-plane flexure; if such in-plane flexure is large enough to result in inelastic rotation, the strength and stiffness with respect to out-of-plane flexure may be significantly reduced.

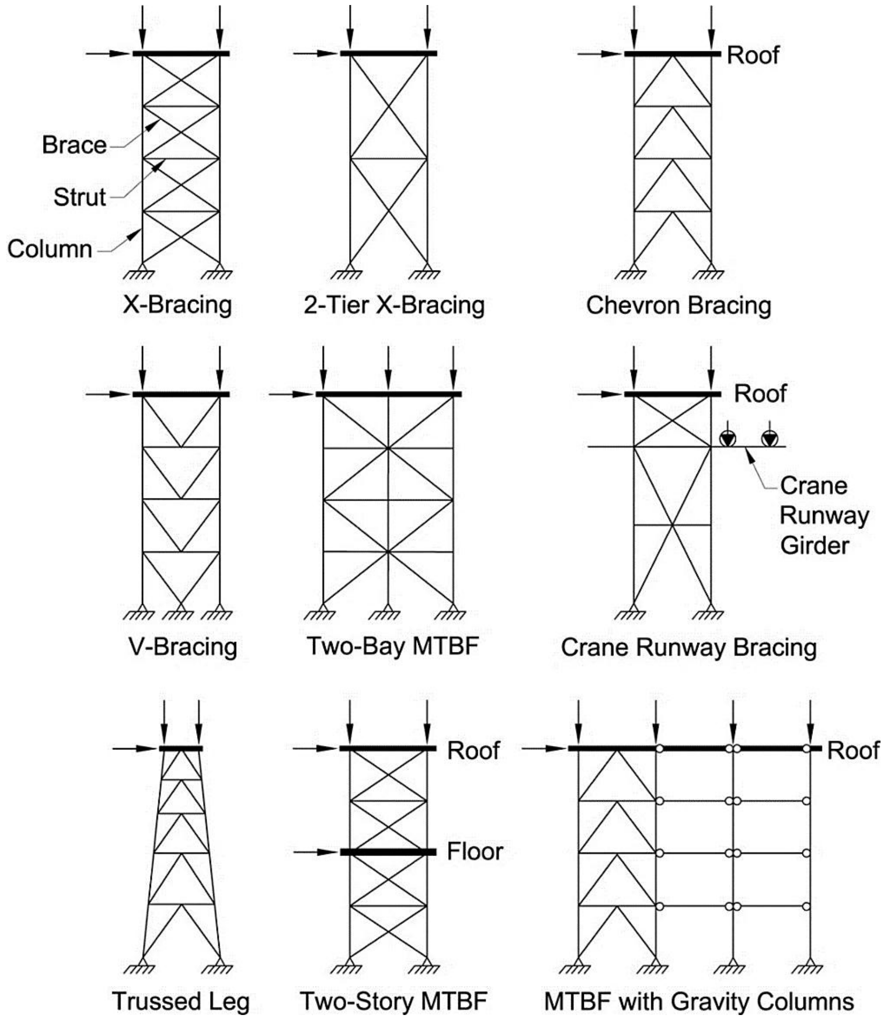


Figure 4 - Multi-tiered Braced Frames (AISC 341, 2016)

To address this effect, AISC Seismic Provisions (AISC 341-16) Section F1.4c includes provisions for the design of multi-tiered OCBF. While design of multi-tiered EBF is possible, the AISC Seismic Provisions do not include a procedure due to the possible complex interactions of unbraced links and unbraced columns. The design of multi-tiered OCBF focuses on increased required strength to reduce the probability of the inelastic column rotation demands.

OCBF's are designed to remain in the elastic range during a seismic event. As a result, the special requirements for the design of OCBF's are relatively few when compared to other steel systems that are required to be designed as more ductile such as SCBF.

The design of OCBF systems is addressed in AISC Seismic Provisions Section F1. They anticipate little inelastic deformation and are designed using a higher seismic force level to account for their limited system ductility. Aside from a few limitations specified by the AISC Seismic provisions, all the components of an ordinary braced frame are designed using the steel design procedures as outlined in the AISC 360 Specification.

According to ASCE 7-16, the use of OCBF's is limited to building heights less than 35 ft for seismic design categories D and E and not permitted in seismic design category F because of their lack of ductility and overall seismic robustness. Therefore, they are an attractive choice for smaller buildings and nonbuilding structures, due to their relatively simpler design and construction procedures. But for the larger buildings and buildings that require a better (more ductile) seismic performance objective, OCBF systems are less desirable.

To ensure that the structural system performs as intended under seismic loadings, some members must be designed to sustain the full, or actual seismic force V_y as adjacent member's yield. These members are referred to as force-controlled members. The design seismic force can be modified by the overstrength factor Ω_o to calculate V_y. According to FEMA 450: NEHRP Recommended Provisions commentary (FEMA 450, 2003), the Overstrength Factor Ω_o, is described by three system specific criteria: design overstrength, material overstrength and system overstrength. Design overstrength is directly related to a structural system's ductility and site-specific ground motion criteria. The material overstrength reflects the actual material strength which exceeds the nominal design material strength. It is also an indication to the structural system redundancy. All these three overstrength criteria are accounted for as a single value Ω_o which is then used to modify the design elastic seismic forces (NEHRP Recommended Provisions, 2003).

1.3 Requirements of OCBF systems in AISC Seismic Provisions, AISC 341

For detailing and designing the OCBF systems, there are a few special considerations. The design of OCBF members is mostly based upon typical steel design procedures as outlined in the AISC 360 Specification and additional requirements given in the AISC 341. One of the important seismic requirements for the design of OCBF connections is the determination of design forces. The AISC Seismic Provisions, AISC 341, specifies that diagonal brace connections must be designed for an amplified seismic load effect using the overstrength factor Ω_o.

The requirements of OCBF systems in AISC Seismic Provisions include the following:

a) Braces are moderately ductile members as given in Section F1.5a, except in frames with tension-only braces that have slenderness ratios greater than 200.
b) The required strength of bracing connections is given in Section F1.6a, which is intended to protect the connection as the brace approaches yielding or buckling, thus providing improved ductility for the system.
c) The brace slenderness limit of $(L_c/r) \leq 4*\sqrt{(E/f_y)}$ for V- or inverted-V configurations is given in Section F1.5b.
d) The requirements for beams in V- or inverted-V frames are given in Section F1.4a.
e) The required strengths of beam and their connections are to use the overstrength seismic loads as given in AISC Seismic Provisions Section F1.5c.

The primary members within a braced frame for the transfer of lateral forces are braces. They resist axial loads in pure axial tension or compression. Once the load has been determined from the load combinations including seismic loads, the member design can be performed. For this study, a wide flange section was used for the brace member for first example, and a Hollow Structural Section (HSS) was for the brace member for second example. First, member stability requirements including the slenderness and local buckling requirements were established and then the strengths of the selected members were calculated.

Typically, a brace must meet the slenderness criteria given in Chapter E of the AISC 360 Specifications. The limit on the slenderness in the V- and inverted-V- braces is intended to limit the unbalanced forces developed in the braced frame beam when the compression brace buckles, and its strength degrades while the tension brace yields. The slenderness limit does not apply to two-story X braced frames, due to the elimination or reduction in the unbalanced force on the beam. The slenderness limit provided by the AISC Seismic Provisions (AISC 341-16) is significantly smaller than that provided by the AISC 360 Specification, which results in a brace that performs better under the action of compression loads.

Like all the compression members, brace must have cross-section of sufficient dimensions to prevent the local deformations that are detrimental to the global member strength. This is especially critical for a seismic lateral force resisting system, due to the cyclic nature of seismic loads such that the brace experiences tension and compression. Furthermore, a brace that buckles locally under the action of compression loads will experience a significant strength degradation. Therefore, AISC Seismic Provisions (AISC 314-16) exceeds the compactness or width-to-thickness ratio requirements of the AISC 360 Specification as given in Table B4.1.

The tensile strength of the brace, like its compression strength, follows the procedure given in Chapter D of the AISC 360 Specification. Tensile yielding must be calculated and, depending on the layout of the connection, tension rupture must also be computed to determine the critical tensile strength based on the equations in Section D2 of AISC 360-16.

The gusset plates are critical elements for resisting the applied forces from the braces and transmitting them to the adjacent members. They must resist shear rupture and yielding due to tension load and compression buckling due to compression load. In the determination of the gusset plate limit states, the full width of the gusset plate perpendicular to the load is not necessarily effective in resisting tension or compression forces from the braces. Therefore, only the area that is effective in resisting the force should be used in the calculations regarding the cross-sectional area of the gusset plate. This area is determined by calculating the Whitmore Section to determine the effective width. The length of the section is equal to the length of the joint. For bolted connection, the length is equal to the center-to-center distance between the first and last bolt lines. The width of the Whitmore section is defined by two lines projected from either side of the start of the joint at a 30-degree angle. Example of the Whitmore section for a bolted connection is shown in Figures 5.

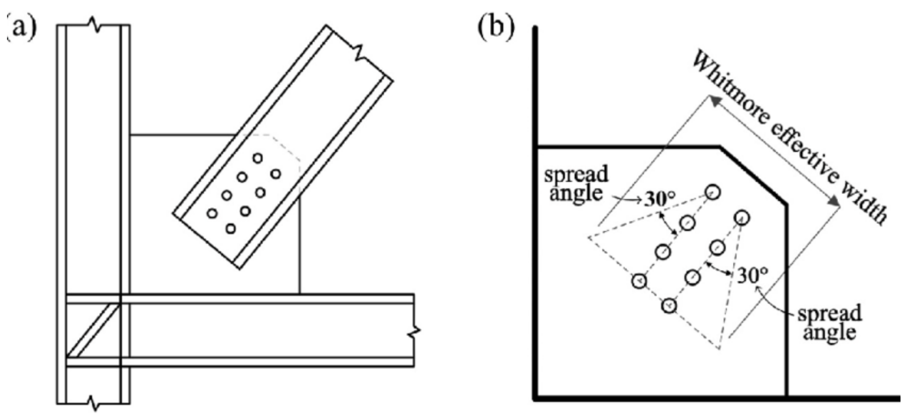


Figure 5 - Whitmore section for a gusset plate with bolted connection (Wang et al., 2023)

The design of columns in an OCBF does not have any special requirements prescribed by the AISC Seismic Provisions, AISC 341-16, because they will perform elastically during the Maximum Considered Earthquake (MCE_R). Therefore, the column design follows the design requirements as specified by AISC 360-16. The major difference in column design in a braced frame is the additional forces applied to the column depending on the brace layout.

Beam in an OCBF must resist the combined effects of bending, compression, and shear. The AISC Seismic Provisions, AISC 341-16, requires that the assumed force in the tension brace be taken equal to its expected yield strength which is computed based on the equation ($R_yF_yA_g/\ \alpha_s$) as given in section F5.6a of AISC 341-16, where α_s is the force adjustment factor, which is 1 for LRFD method and 1.5 for ASD method.

Similarly, in compression, the expected brace strength in compression is divided by α_s which is permitted to be taken as the lesser of ($R_yF_yA_g/\ \alpha_s$) and ($1.1F_{cre}A_g/\ \alpha_s$) as given in section F5.6b of AISC 341-16, where F_{cre} is determined from Chapter E of the AISC 360-16 Specifications using the equations for F_{cr}, except that the expected yield stress , R_yF_y is used in lieu of F_y.

Once the beam forces are determined based on analysis and the AISC Seismic Provisions, AISC 341-16, then beams, columns and connections can be designed in accordance with the AISC Specification. The beam must meet the compactness requirements of Table B4.1 (AISC 360-16) and Section F2 (AISC 360-16) to determine the flexural strength of the beam. The beam IN OCBF does not need to be a seismically compact because like the columns, the beam is expected to remain elastic during the Maximum Considered Earthquake (MCE_R). Since the beam is subjected to combined bending and compression forces, the combined interaction according Chapter H of AISC 360-16 must also be checked.

For gusset-beam connection, the forces at the interaction plane are determined using the uniform force method as outlined in Chapter 13 of the AISC manual. Both vertical and horizontal forces are applied to the beam by the gusset plate and the weld must be designed for that interaction. Additionally, the limit states that must be checked at the gusset-to-beam connection are gusset plate rupture, gusset plate yielding, beam web local yielding and beam web crippling.

Similarly, for the gusset-to-column connection the procedure followed is the same as that of gusset-to-beam connection.

The beam-to-column connection must be designed for both – gravity loads applied to the beam and the vertical and horizontal components of the brace forces using the applicable load combinations as per ASCE7-16.

1.4 Component Based Finite Element Method (CBFEM)

Currently, several existing programs are based on the Component method which can be used for the solution. But the major drawback here is that for each topology, a new model must be created for each case. IDEA StatiCa is based on a new method known as Component-based Finite Element Model (CBFEM) which is a synergy of component method and finite element method (Wald,et al., 2015, 2021). The design-oriented finite element model (DOFEM) using CBFEM is extensively verified, and several studies are published and it is implemented in IDEA StatiCa and Hilti PROFIS (Šabatka et al., 2020).

Design-oriented finite element analysis for steel connections is carried out with the IDEA StatiCa connection. The software combines the finite element method with the component method and offers an alternative to conventional analytical models and the laborious component method. Contrary to RFEM, IDEA software uses 2D shell elements for plates, whereas fasteners (welds, bolts, contacts, etc.) are represented by components with pre-defined properties based on experimental findings (Kuříková et al., 2021).

CBFEM also provides design code checks of failure modes that are very difficult to capture by finite element analysis alone, such as the crushing of concrete in compression or weld fracture. It removes the restrictions and most simplifications used in the Component Method. The neutral axis and forces in components for any type of load combination are determined by the finite element method (Šabatka et al., 2020).

• Joint are divided into individual components as shown in Figure 6 (IDEA StatiCa, n.d.-c; Wald,et al., 2021).

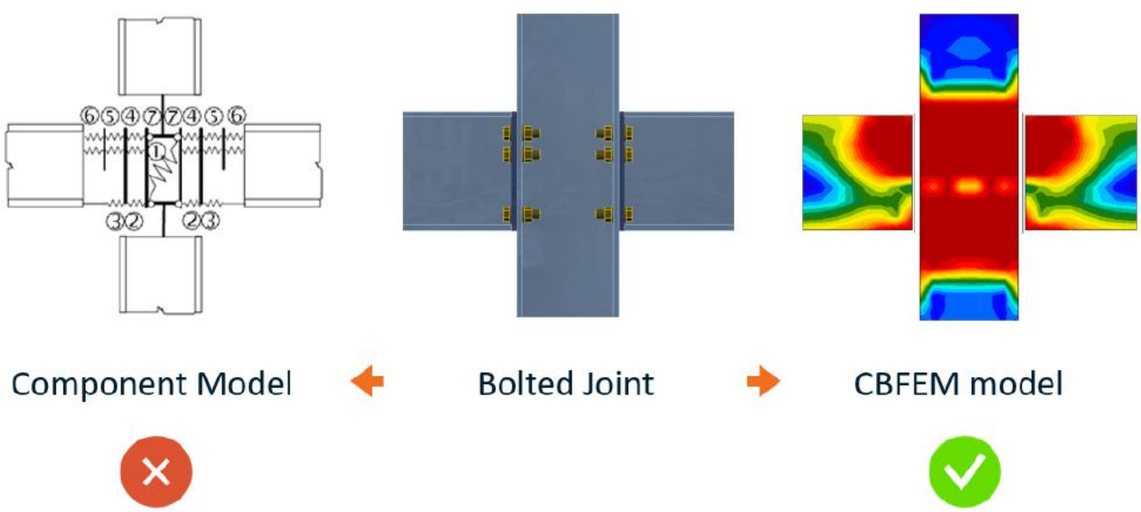
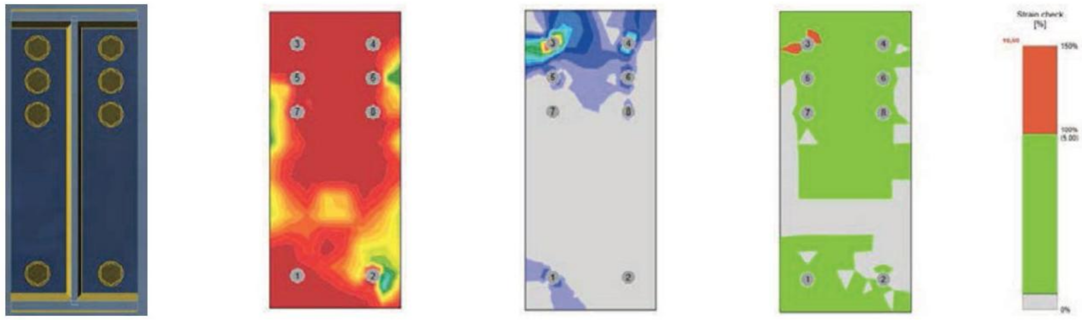


Figure 6 - Components of a beam-to-column joint and a simple spring model(Block et al., 2012; IDEA StatiCa, n.d.-c)

- All steel plates are modelled by Finite Element Method as shell elements assuming ideal elastic-plastic material as shown in Figure 7 (IDEA StatiCa, n.d.-c).



3D model   Equivalent stress   Equivalent strain   Overall check

Figure 7 - Steel plates as shell elements in CBFEM (IDEA StatiCa, n.d.-c)

• Bolts are modelled as nonlinear springs as shown in Figure 8 (IDEA StatiCa, n.d.-c)

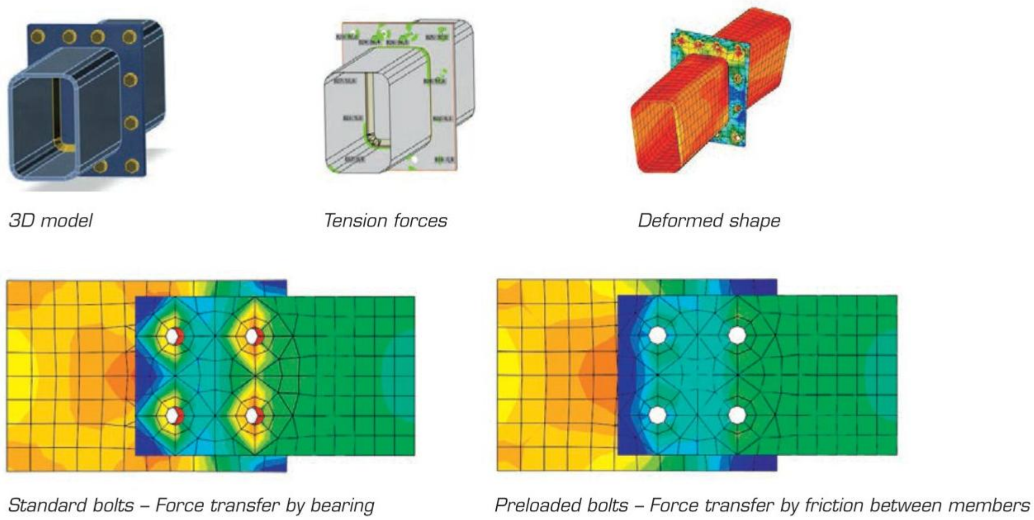
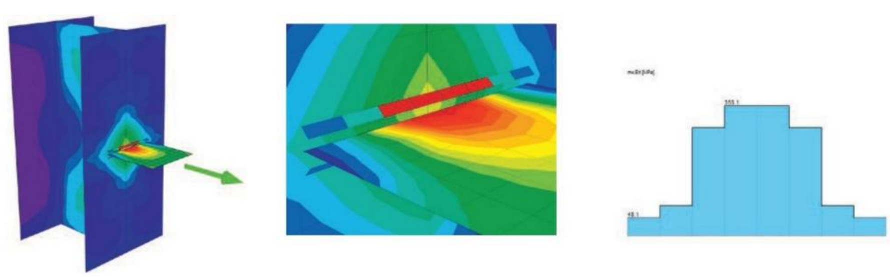


Figure 8 - Standard bolts and Preloaded bolts in CBFEM (IDEA StatiCa, n.d.-c)

- Welds are modelled as elastoplastic shell elements to allow for plastic redistribution as shown in Figure 9 (IDEA StatiCa, n.d.-c)



3D visualisation of Equivalent stress. Output table with results. Equivalent stress in weld.

$$UT = \max\left(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}}; \frac{|\sigma_{\perp}|}{\sigma_{\perp,Rd}}\right) = \quad 98.1 \quad \%$$

$$\sigma_{w,Ed} = [\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)]^{0.5} = \quad 353.1 \quad \text{MPa}$$

$$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) = \quad 360.0 \quad \text{MPa}$$

$$\sigma_{\perp,Rd} = 0.9 f_u/\gamma_{M2} = \quad 259.2 \quad \text{MPa}$$

where:

f_u = 360.0 MPa – Ultimate strength

γ_{M2} = 1.25 – Safety factor

Status	Item	Edge	Th [mm]	L [mm]	Loads	σw,Ed [MPa]	ε,Pl [%]	σ⊥ [MPa]	τ|| [MPa]	τ⊥ [MPa]	Ut [%]	Utc [%]
✓	C-bfl 1	B-bfl 1	◢4.0◣	100	LE1	353.1	0.2	160.9	-84.0	160.9	98.1	53.8

Result tables for the weld

Formulas and values for the weld

Figure 9 - Welds in CBFEM (IDEA StatiCa, n.d.-c)

The finite element model is used for analyzing internal forces in each of the components (IDEA StatiCa, n.d.-c; Wald,et al., 2021). Plates are checked for limit plastic strain – 5 % acc. to EC3, which corresponds well also with recommendations in AISC 360-16 for rotation limit 0.02 rad (IDEA StatiCa, n.d.-c; Wald,et al., 2021). Each component is checked according to specific formulas defined by the national code, similar to when using Component Method (IDEA StatiCa, n.d.-c). The stiffness of the joint is determined by finite element analysis (IDEA StatiCa, n.d.-c). The member sections are decomposed into plates using shell elements with 6 total degrees of freedom at each joint (3 rotations and 3 translations). Material non-linearity analysis was performed to overcome challenges posed by the discontinuous regions in the models (IDEA StatiCa, n.d.-c).

IDEA StatiCa Connection focuses on the actual checking (and not the analysis) of the connection node, thereby providing the engineer with the possibility to overcome the limitations of the current Component method, on which all the other existing connection software are based. It also provides a simple passed/failed status for all elements, which is something the general FEM tools simply do not have. (IDEA StatiCa, n.d.-b).

For the presented study, two connections- corner connection and two-story X-bracing chevron connection which are common in an ordinary concentrically braced frame (OCBF) system are analyzed using CBFEM and verification with AISC is performed.