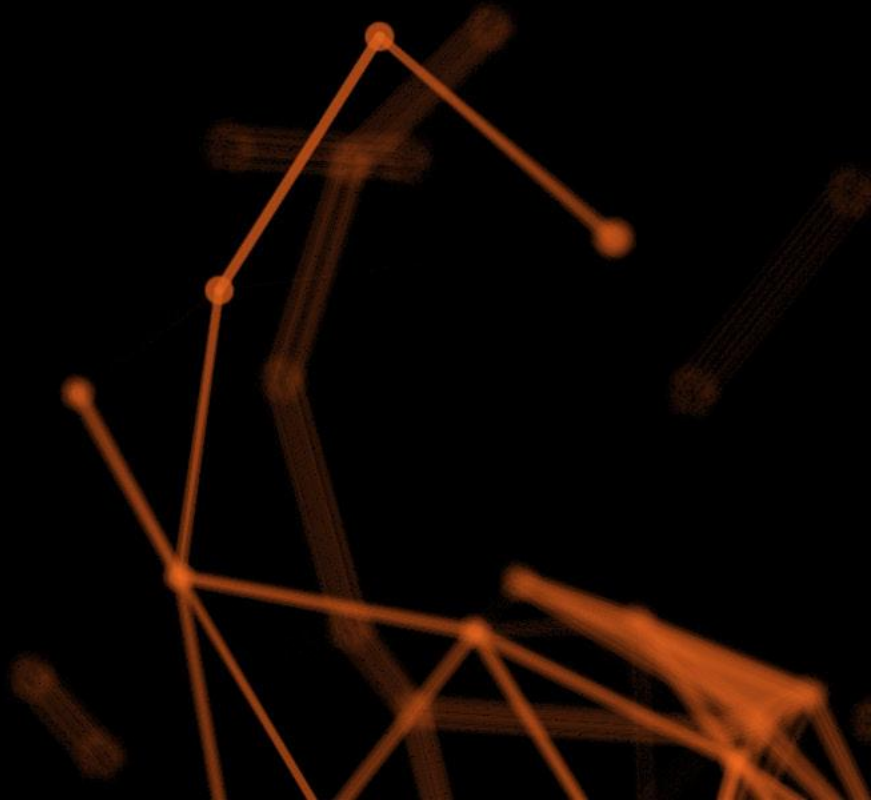
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Calculate yesterday's estimates

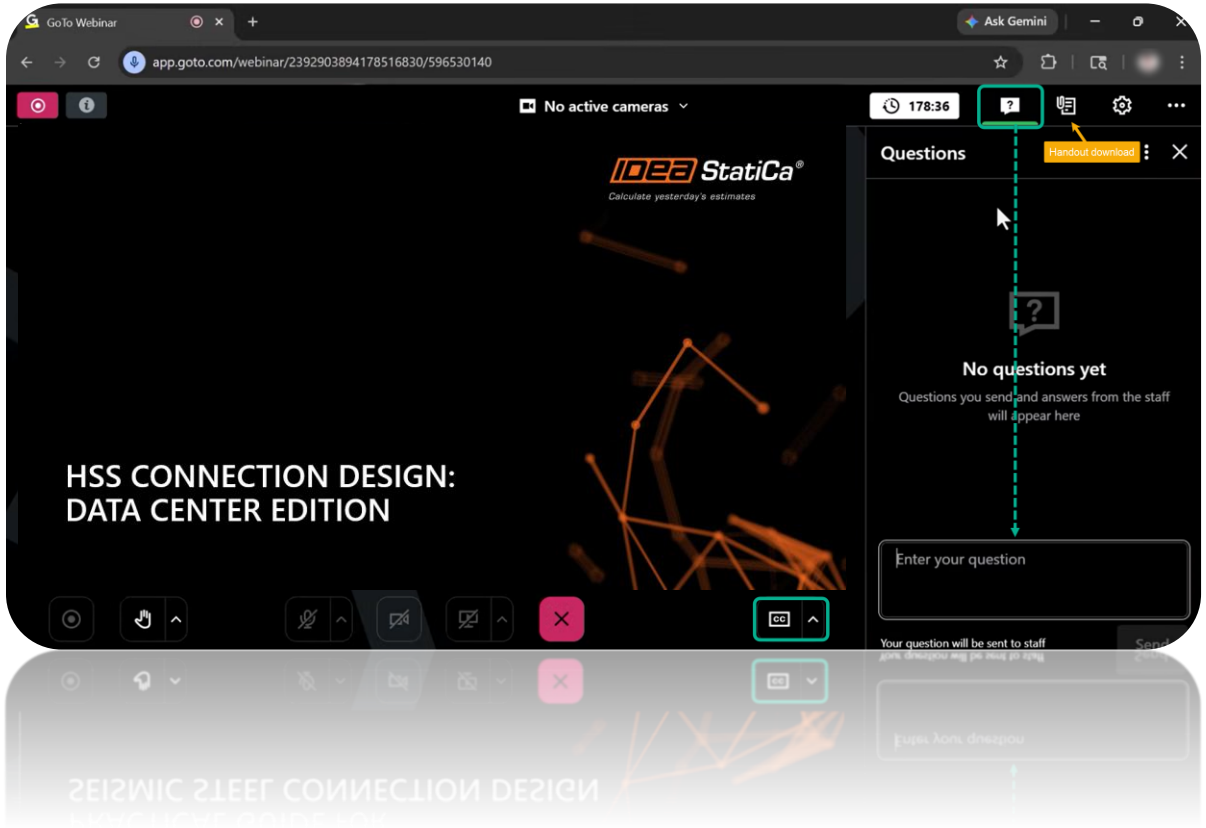
HSS CONNECTION DESIGN: DATA CENTER EDITION



GOTO WEBINAR CONTROL PANEL

1. To ask a question, select the **Questions** pane, then type your question and select **Send**.
2. To view any shared handouts, select the **Handouts** pane, and then select the desired handout.
3. Recording and certificate will be sent **2 days later**

❖ If you have audio issues, try closing and re-opening GoTo Webinar app

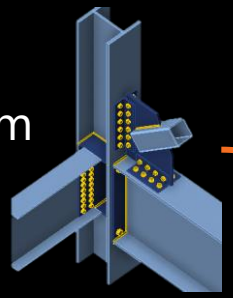


AGENDA

- Intro to CBFEM & GMNA
- Southern Steel Engineers - Data center case study
- Live demonstration – HSS connection results overview
- Q&A session

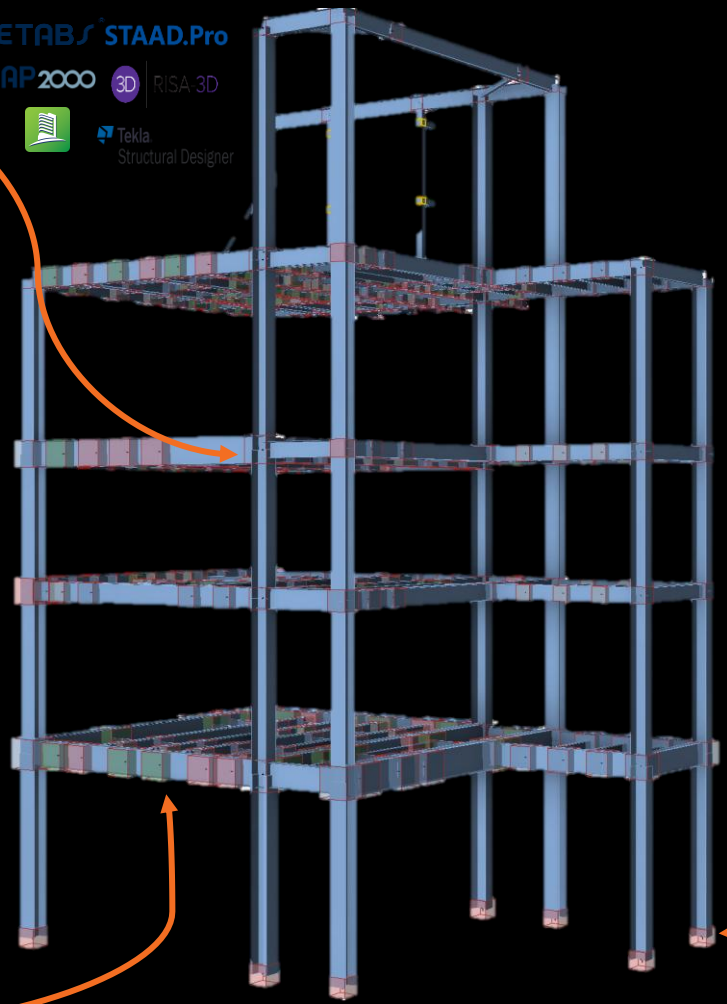
Connection

Design all steel connections from standard to complex joints



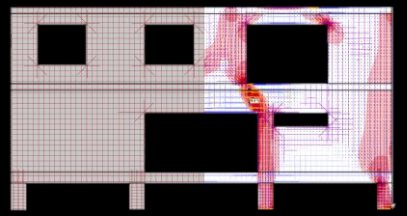
Checkbot

ETABS STAAD.Pro SAP2000 RISA 3D Tekla Structural Designer



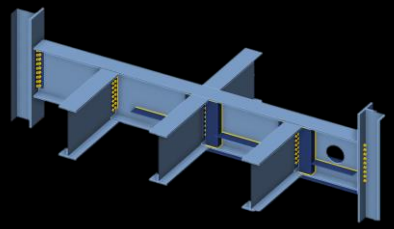
Detail

Strut and tie replacement for:
Transfer beams
Shear walls
Corbels
Walking columns

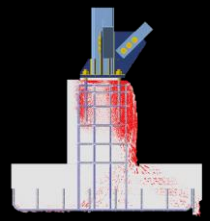


Member

Critical Beam and column design



Connection + Detail



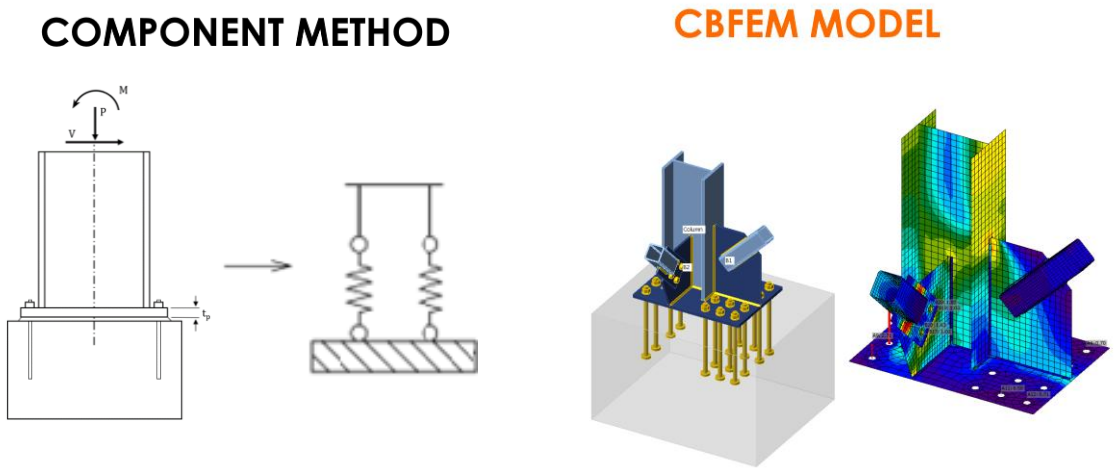
Complete base plate workflow including steel and concrete reinforcement design to avoid concrete breakout

COMPONENT BASED FINITE ELEMENT METHOD

CBFEM is a synergy of the Component Method and Finite Element analysis used in Connection.

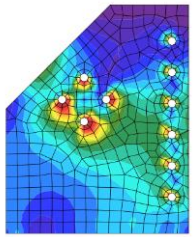
The check itself consists of two steps:

1. Forces in each component of the joint are calculated using FEA
2. Each component (bolts, welds, anchors) is checked using the code equations

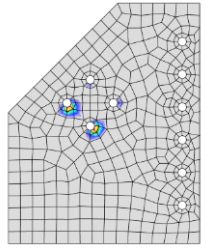


CBFEM CODE CHECKS OF STEEL CONNECTION COMPONENTS

Plates: Material properties are checked, design limit is based on plastic strain



Equivalent stresses



Plastic strain

Check of members and steel plates for extreme load effect

	Status	Item	Grade	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]
−	✓	GUSS1	A572 Gr.50	1/2	LE1	45.7	2.2	3.9

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

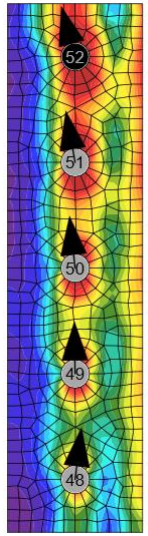
Where:

$F_y = 50.0$ ksi – characteristic yield strength

$\phi = 0.90$ – resistance factor for steel material AISC 360-22 – B3.1

Bolts and anchors

AISC Code equations



Tension resistance check (AISC 360-22 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 29.79 kip ≥ $F_t =$ 9.31 kip

Where:

$F_{nt} = 89.9$ ksi – nominal tensile stress AISC 360-22 – Table J3.2

$A_b = 0.4418$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Shear resistance check (AISC 360-22 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 17.88 kip ≥ $V =$ 5.97 kip

Where:

$F_{nv} = 54.0$ ksi – nominal shear stress AISC 360-22 – Table J3.2

$A_b = 0.4418$ in^2 – gross bolt cross-sectional area

$\phi = 0.75$ – resistance factor

Bearing resistance check (AISC 360-22 – J3-6)

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

$\phi R_n =$ 25.21 kip ≥ $V =$ 5.97 kip

Where:

$l_c = 1"1/8$ in – clear distance, in the direction of the force, between the edge hole or edge of the material

$t = 3/8$ in – thickness of the plate

$d = 3/4$ 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

Interaction of tension and shear check (AISC 360-22 – J3-3)

$\phi R_n = \phi \cdot F'_{nt} \cdot A_b =$ 28.78 kip ≥ $F_t =$ 9.31 kip

Where:

$F'_{nt} = 86.9$ ksi – nominal tensile stress modified to include the effects of shea

- $F'_{nt} = 1.3 \cdot F_{nt} - \frac{f_{rv} \cdot F_{nt}}{\phi \cdot F_{nv}} \leq F_{nt}$, where:
  - $F_{nt} =$ 89.9 ksi – nominal tensile stress AISC 360-22 – Table J
  - $F_{nv} =$ 54.0 ksi – nominal shear stress AISC 360-22 – Table J3
  - $f_{rv} =$ 13.5 ksi – required shear stress using LRFD or ASD loa of the fastener shall be equal or exceed the required shear stres
  - $\phi =$ 0.75 – resistance factor for tension and shear combinatio

$A_b = 0.4418$ in^2 – gross bolt cross-sectional area

Welds

AISC Code equations

Check of welds for extreme load effect

	Status	Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Ut_c [%]	Detailing
+	✓	BP1	W14X90-tfl 1	E70xx	◢ 5/16 ◣	◢ 7/16 ◣	1'-2"1/2	1"3/16	L1T+S+M	16.87	17.02	99.1	99.1	✓
+	✓			E70xx	◢ 5/16 ◣	◢ 7/16 ◣	1'-2"1/2	1"3/16	L1T+S+M	16.38	16.94	96.6	96.6	✓
−	✓			E70xx	◢ 5/16 ◣	◢ 7/16 ◣	1'-2"1/2	1"3/16	L2 C+S+M	13.46	16.83	80.0	69.5	✓

Weld resistance check (AISC 360-22 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 16.83 kip ≥ $F_n =$ 13.46 kip

Where:

$F_{nw} = 60.1$ ksi – nominal stress of weld material:

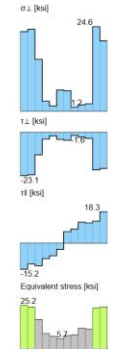
- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 65.1° – angle of loading measured from the weld longitudinal axis

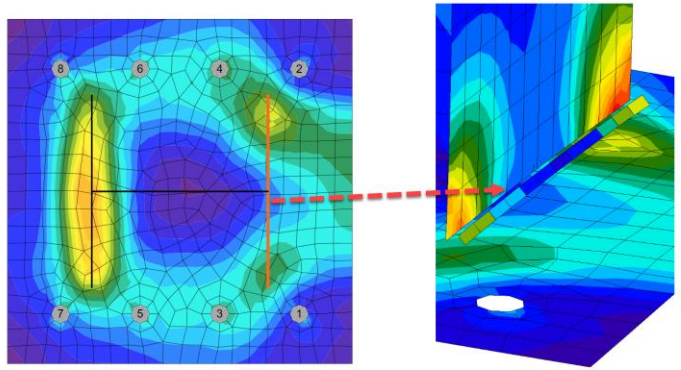
$A_{we} = 0.3732$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Design data

Material	F_{exx} [ksi]
E70xx	70.0



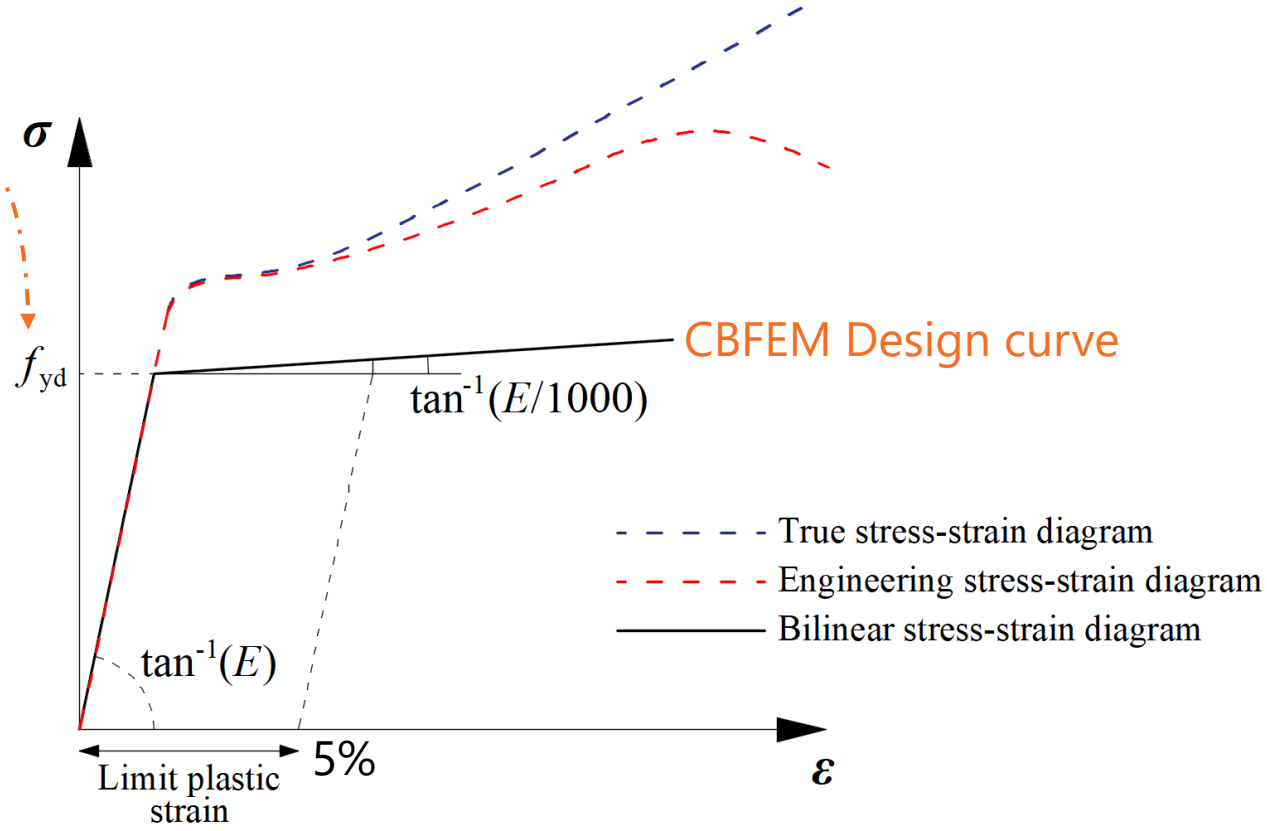


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STEEL MATERIAL MODEL FOR PLATES

Design yield stress:

(LRFD): $f_y * 0.9$ | (ASD): $\frac{f_y}{1.67}$

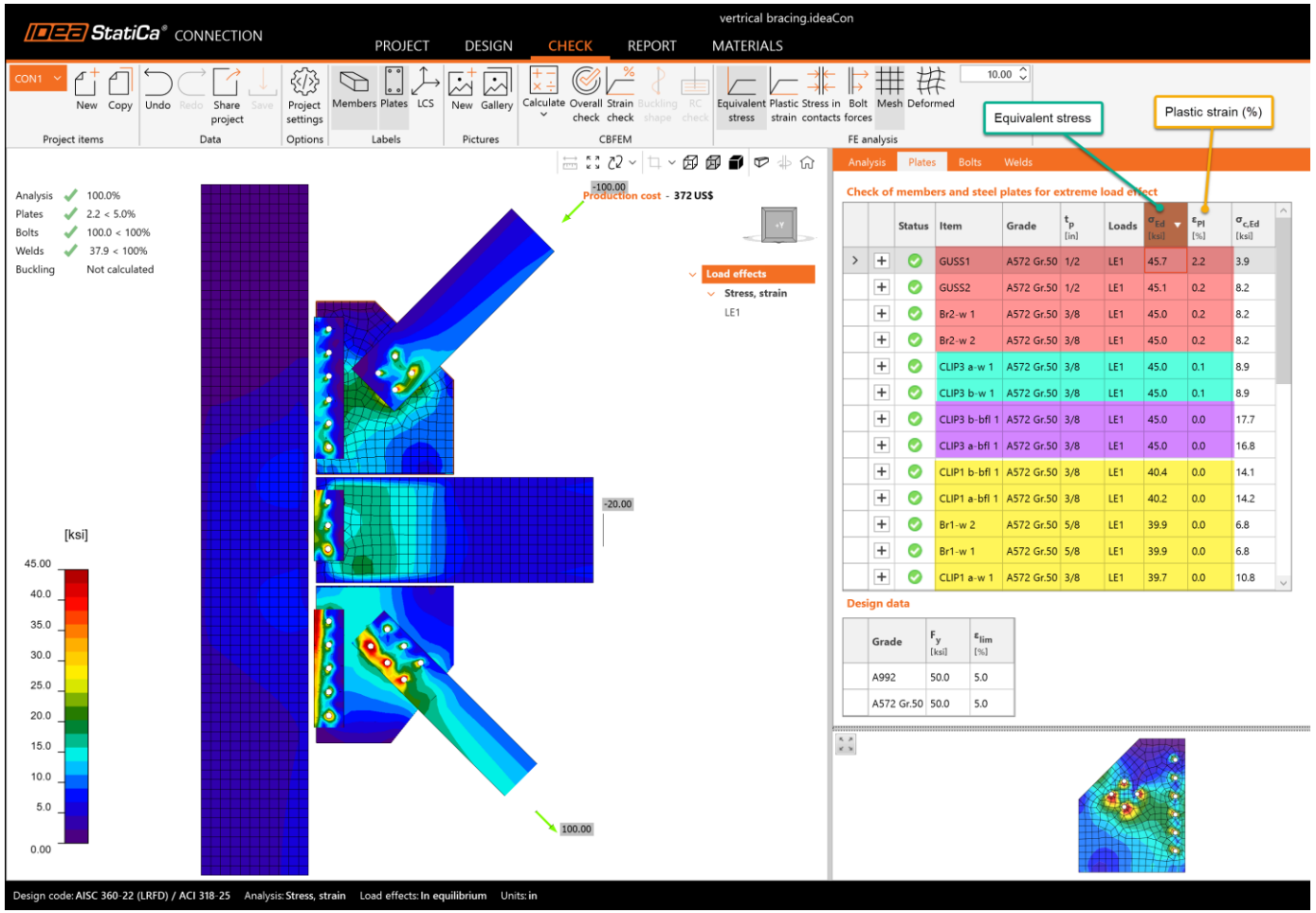


The material behavior is based on the von Mises yield criterion

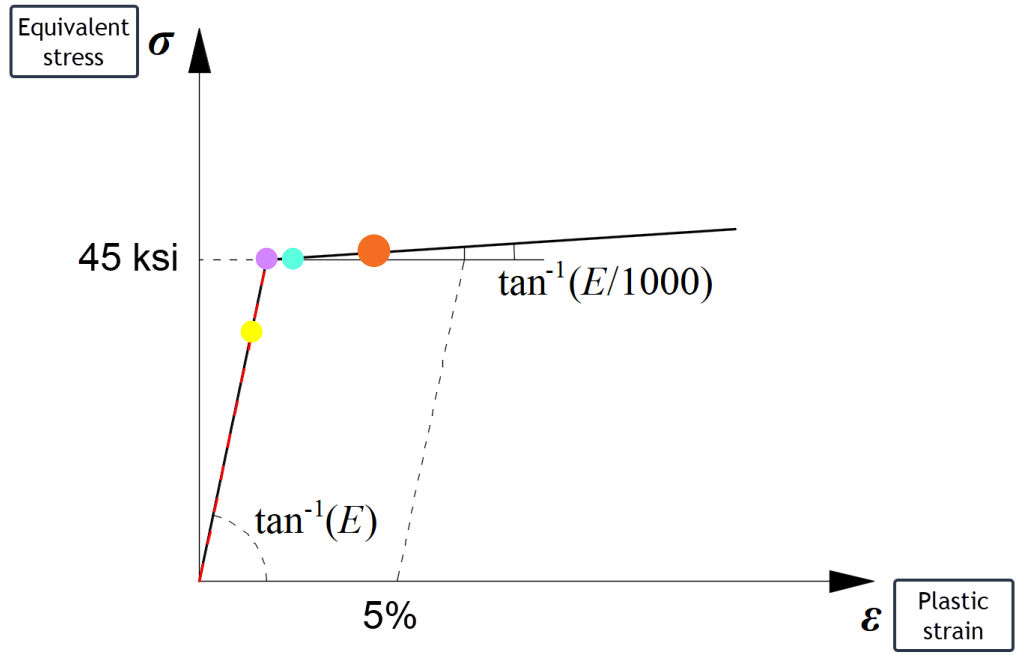
It is assumed to be elastic before reaching the design yield strength, f_{yd}.

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CONNECTION DESIGN (AISC LRFD)



Design yield strength (LRFD) =
50 ksi *0.9 = 45 ksi

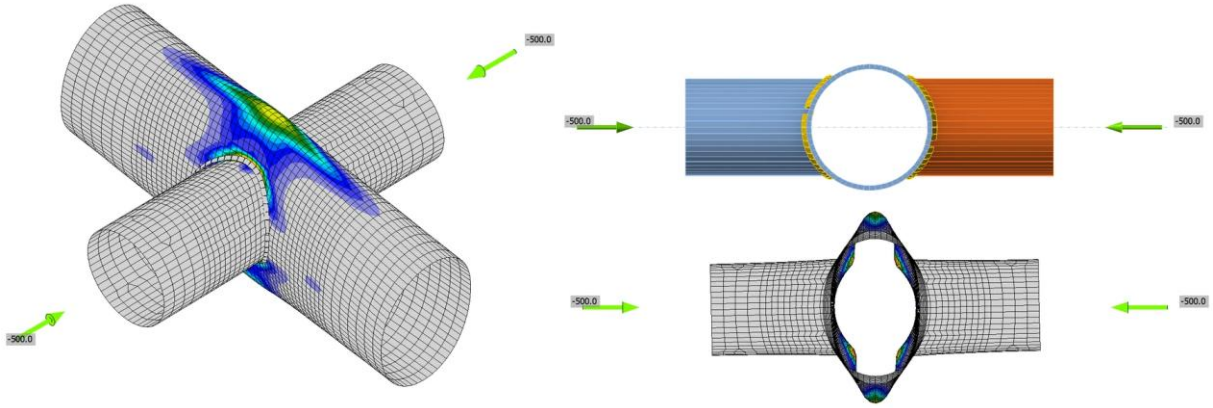


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STRESS-STRAIN ANALYSIS + GMNA

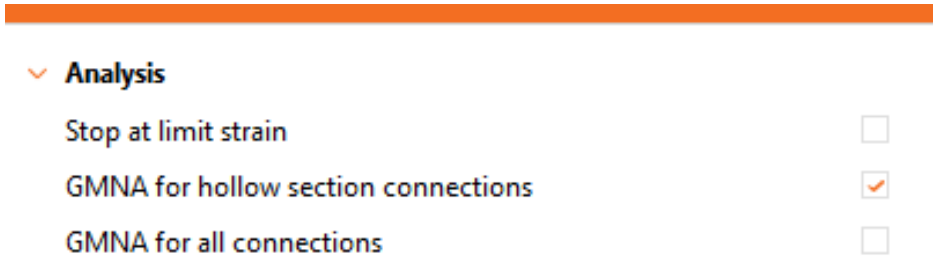
Theoretical Background

Hollow sections: Out-of plane deformation + Geometrical nonlinearity (GMNA)



The geometrical non-linear analysis applies to the nonlinear deformation of certain profile sections like hollow sections.

- GMNA is only used if the bearing member is a hollow section by default.
- GMNA can also be set for all connections under settings

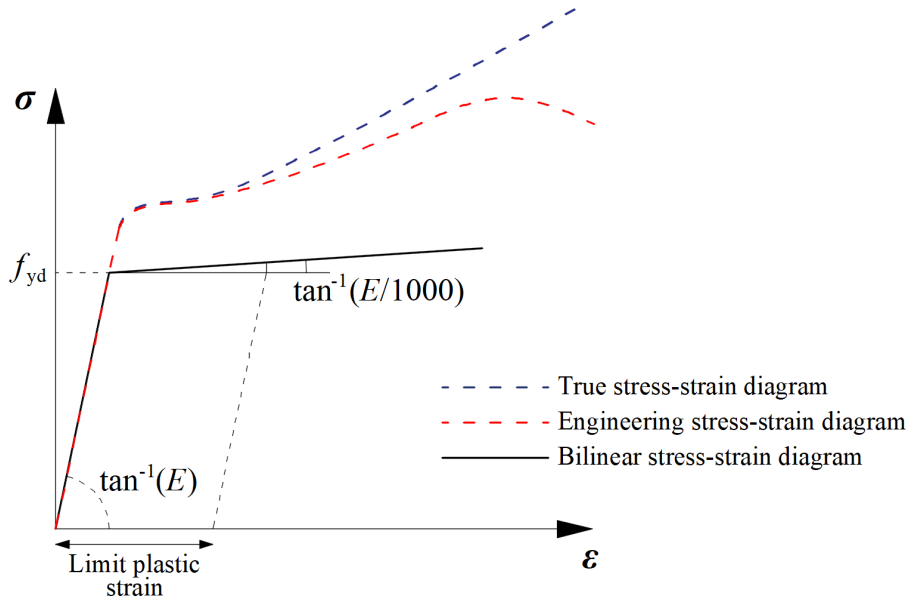


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NONLINEAR ANALYSIS IN IDEA STATICA

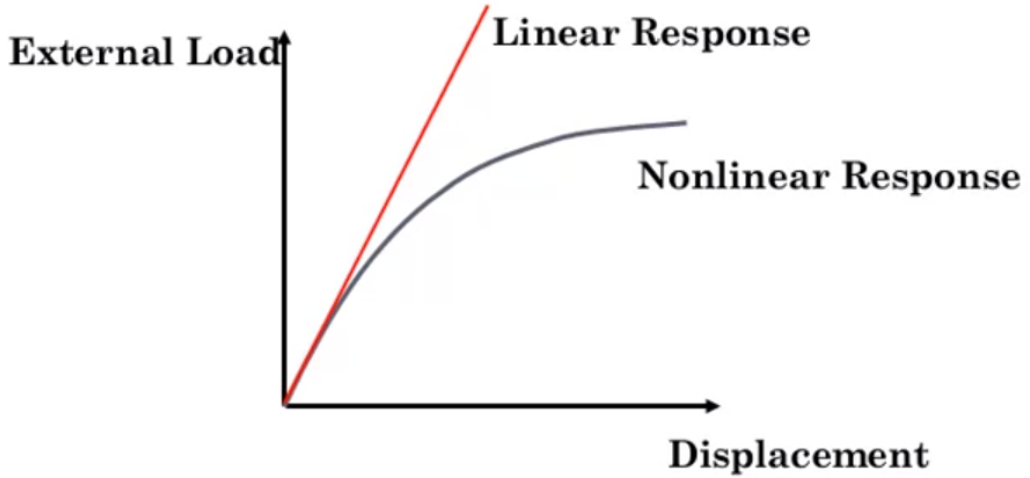
Material nonlinearity

*Active in all the analysis



Geometrical nonlinearity

*Only active when the bearing member is a Hollow section



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HSS LOCAL DEFORMATION

on CIDECT publications and reports can be obtained from their website: **www.cidect.com**.

Chapter K does not prohibit using joints that fall outside the listed limits of applicability; however, this Specification and Commentary do not provide connection capacities or guidance when doing so. A rational approach to their design is left to the designer. This Commentary gives some insight into the failure modes that should be considered. However, some of the discussions presented later concerning which limit states need to be checked, which can be eliminated, and when they can be eliminated, may or may not apply when outside the limits of applicability. There is also one notable failure mode (local buckling of

the chord face) that has been eliminated from consideration in both the Specification and Commentary due to the fact that, in tests, it did not control the connection strength when staying within the limits. All potential failure modes should be investigated by the designer when working outside the limits of applicability listed in Chapter K.

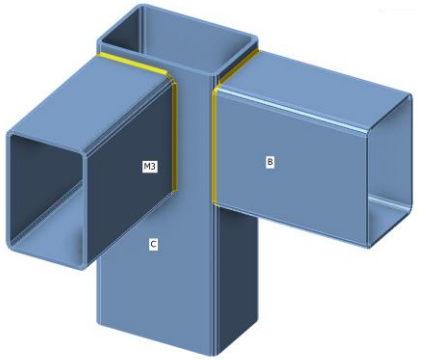
When inelastic finite element analysis is used, peak strains in the thick shell ($T \times T \times T$) elements should not exceed $0.02/T$ at the nominal capacity, where T is the thickness in inches.

The connection capacities calculated in Chapter K are based on strength limit states only. There is no connection deformation limit state considered in these provisions. Subcommission XV-E of IIW, in their most recent design recommendations (IIW, 2012), have now adopted a limit of $0.03D$ for round HSS, where D is the HSS diameter, and $0.03B$ for rectangular HSS, where B is the HSS chord width, as the maximum acceptable connection displacement, perpendicular to the main member face at the ultimate load capacity. This limit state equates to approximately 1% of connection deformation at service loads.

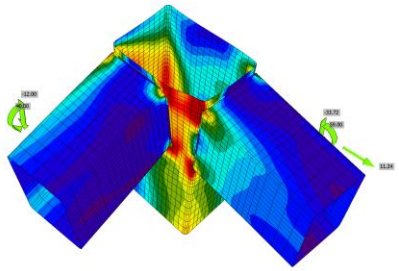
While the majority of Chapter K is in agreement with the previous IIW design recommendations (IIW, 1989), it was determined that adopting a connection deformation limit state for HSS would not be consistent with this Specification, which does not include deformation limit states for connections; however, designers should be aware of the potential for relatively large connection deformations in certain HSS joint configurations. In order to meet the new deformation limit state, IIW and the International Organization for Standardization (ISO) have made some modifications to the range of validity of T, Y, X, and K gap connections and to the calculations of connection strengths, including changes to the strength reduction based on the chord or main member stress function, Q_f. The change in the chord stress function is particularly noticeable in high tension areas of main members where no

AISC Commentary section Ch. K

Isometric view



Iso view
Deformation shape

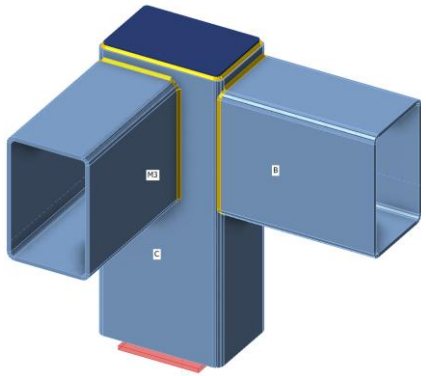


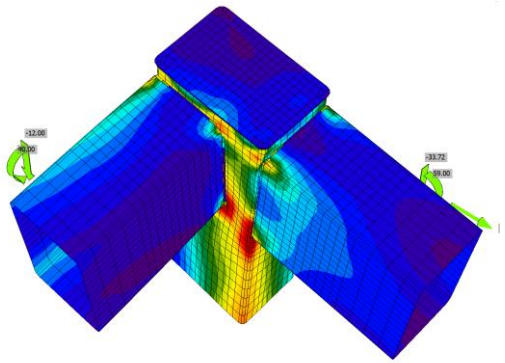
Local deformation check

Analysis | Local deformation | Plates | Welds

Check of members and steel plates for extreme load effect

	Status	Item	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]
>	✓	C	8"	LE1	0.04	0.24	0.4775
	✓	B	8"	LE1	0.01	0.24	0.0805
	✓	M3	8"	LE1	0.01	0.24	0.1108





Analysis | Local deformation | Plates | Welds

Check of members and steel plates for extreme load effect

	Status	Item	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]
>	✓	C	8"	LE1	0.03	0.24	0.4057
	✓	B	8"	LE1	0.00	0.24	0.0000
	✓	M3	8"	LE1	0.01	0.24	0.0960

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CASE STUDY: SOUTHERN STEEL ENGINEERS

Avondale Data Center, Avondale, AZ



IDEA StatiCa®

AVONDALE DATA CENTER: PROJECT OVERVIEW



- Avondale, AZ
- Southern Steel Engineers
- Steel framing structure
- HSS Bracing at exterior screen wall
- Exposed galvanized steel

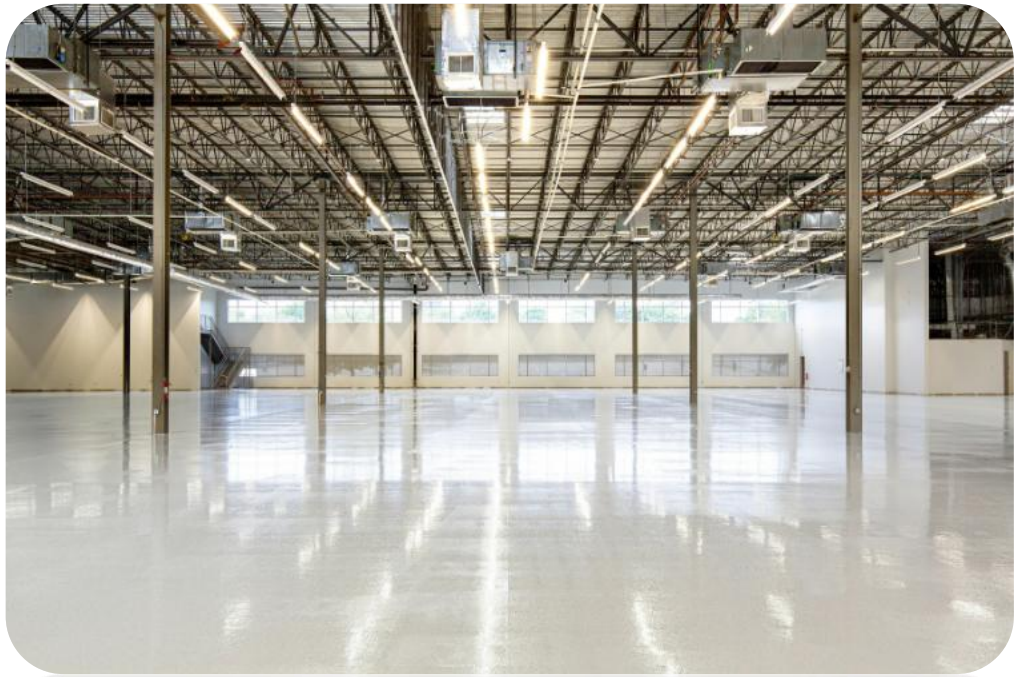
Complex-steel-connection-design-for-hss-bracing-in-a-data-center

SSE SOUTHERN STEEL ENGINEERS

AVONDALE DATA CENTER

Challenges:

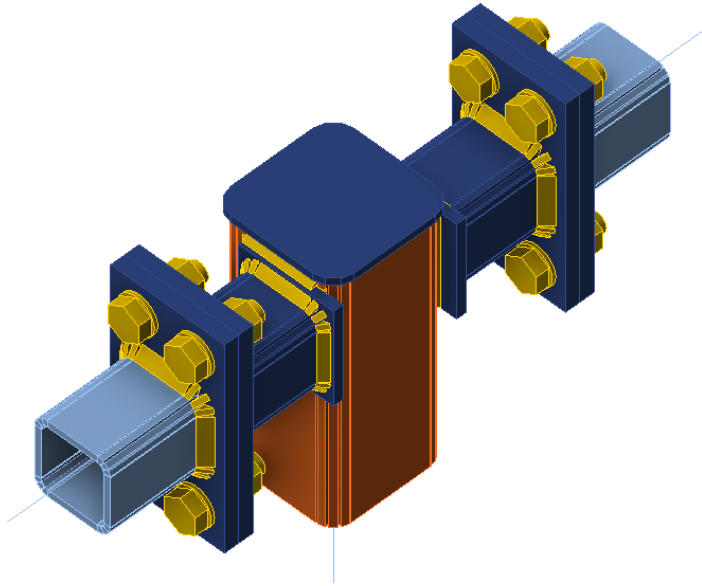
- Combined loading with constrained geometry
- Non-standard HSS bracing connections at exterior screen wall
- All exposed galvanized steel
- Bolted connection preference



"The steel was all galvanized so… this is also the reason that the connections are unique with a desire to try to field bolt as many connections as possible."

SOLUTION: IDEA STATICA CONNECTION

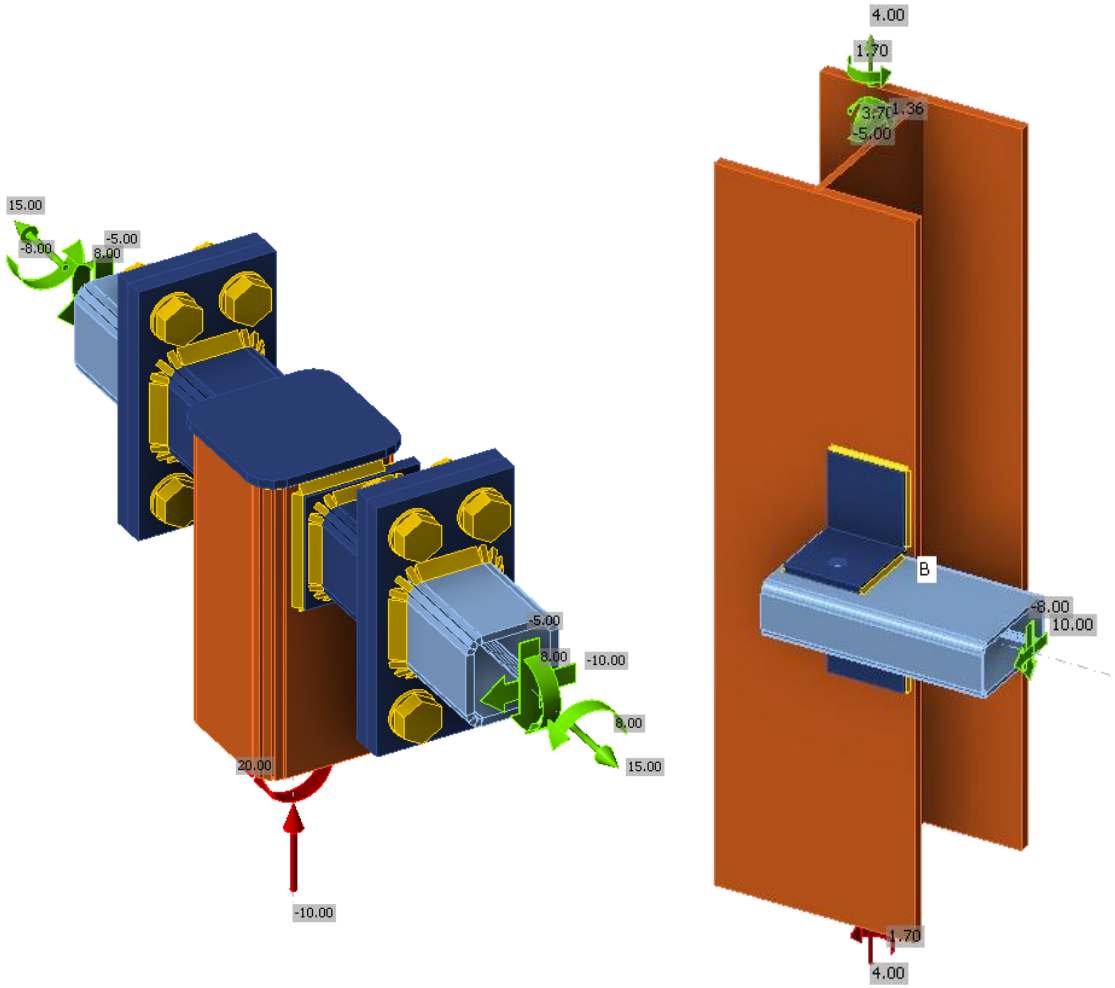
- ✓ Modeled HSS members, gussets, plate to plate, end plates together
- ✓ Combined loading in a single model
- ✓ Fast iteration of brace connection configuration
- ✓ Drainage openings considered in model





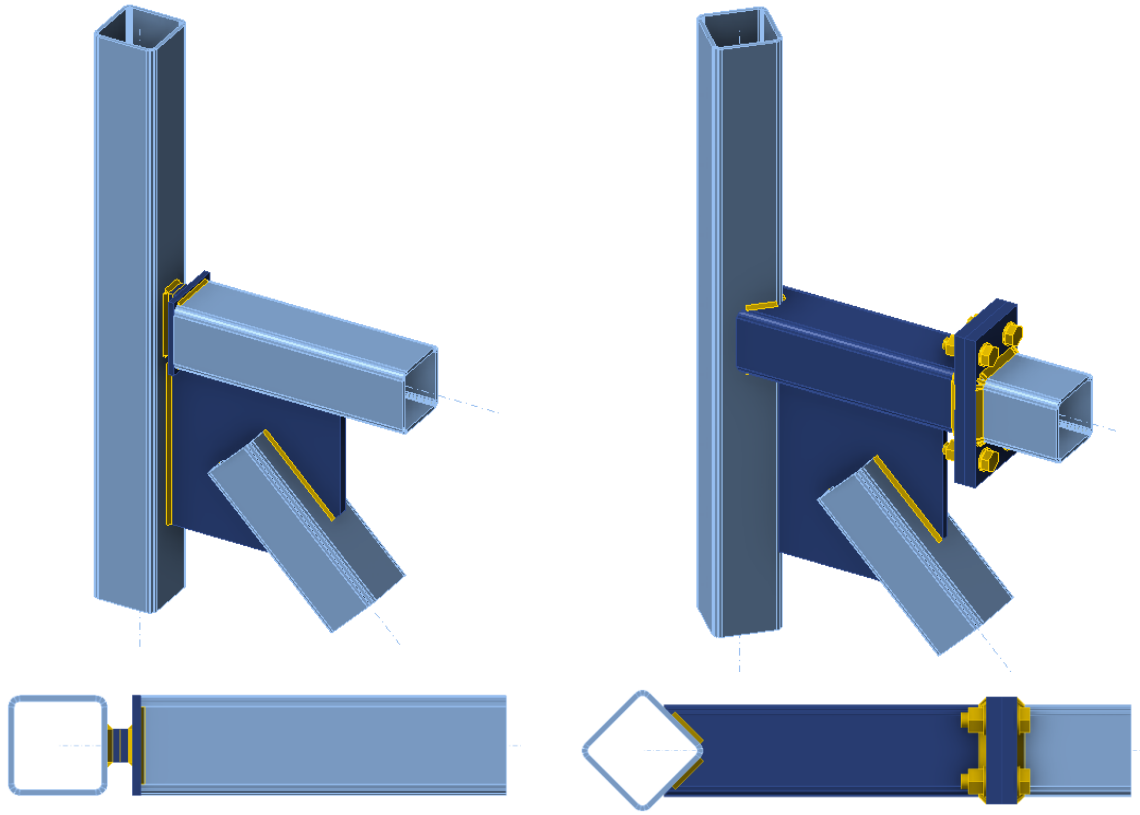
SOLUTION: IDEA STATICA CONNECTION

- ✓ Modeled HSS members , gussets, splices, end plates together
- ✓ Combined loading in a single model
- ✓ Fast iteration of brace connection configuration
- ✓ Drainage openings considered in model



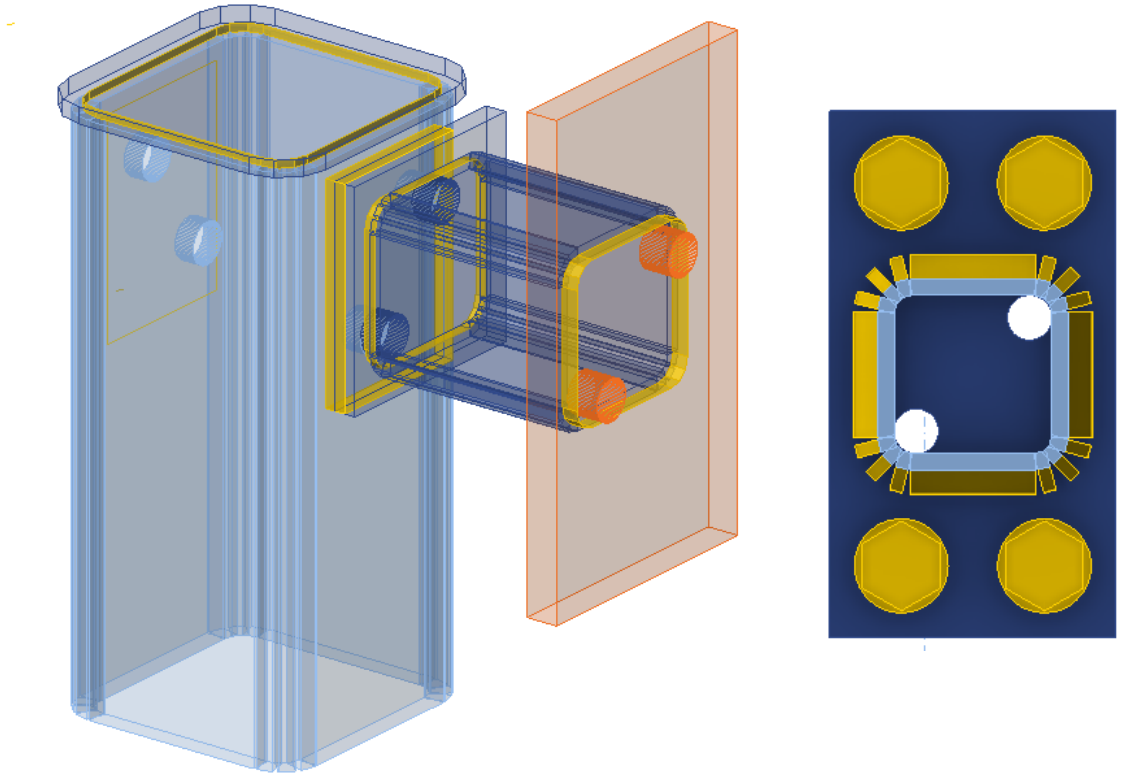
SOLUTION: IDEA STATICA CONNECTION

- ✓ Modeled HSS members , gussets, splices, end plates together
- ✓ Combined loading in a single model
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SOLUTION: IDEA STATICA CONNECTION

- ✓ Modeled HSS members , gussets, splices, end plates together
- ✓ Combined loading in a single model
- ✓ Fast iteration of brace connection configuration
- ✓ Drainage openings considered in model



RESULTS AND IMPACT

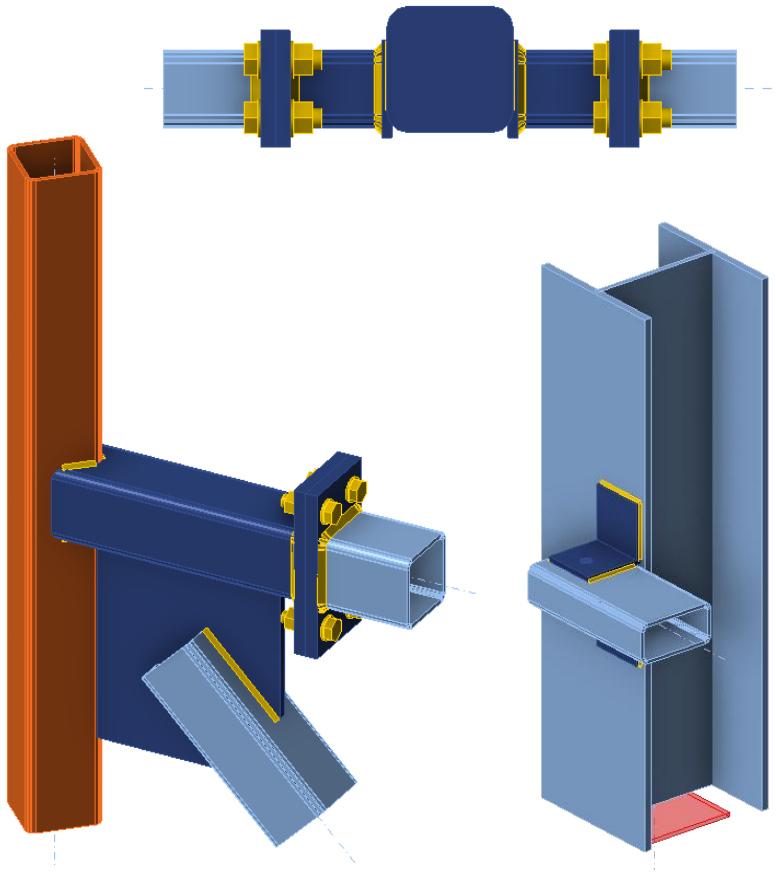
- ✓ Faster validation across connection variants
- ✓ Reduced repetitive manual calculations
- ✓ Field bolting maximized per client preference
- ✓ Galvanization drainage fully integrated

"Being able to model these connections and immediately understand their behavior under load allowed us to move much faster and make informed decisions without overdesigning."

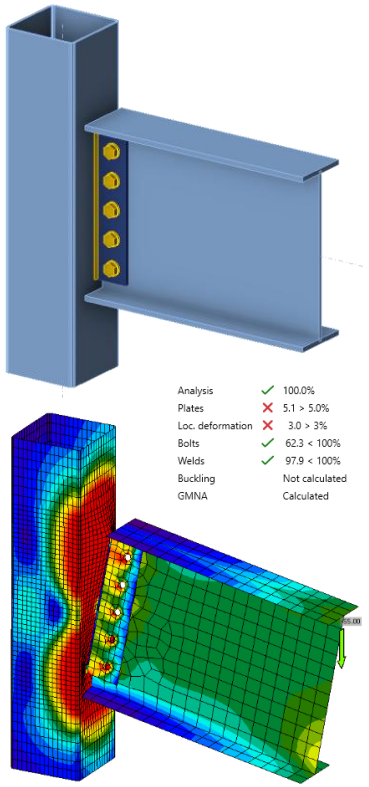
-Southern Steel Engineers

LIVE DEMO

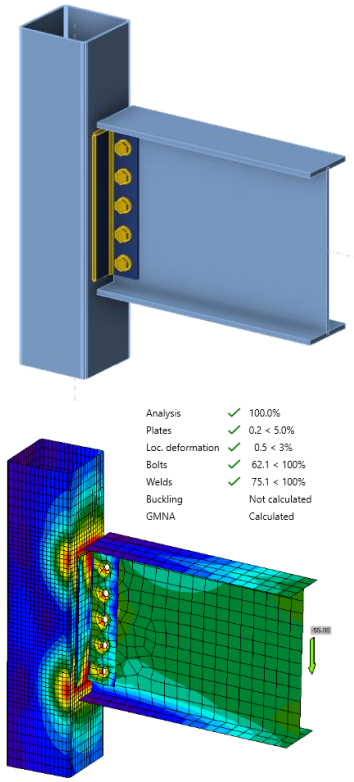
Modeling for constructability



Modeling for design

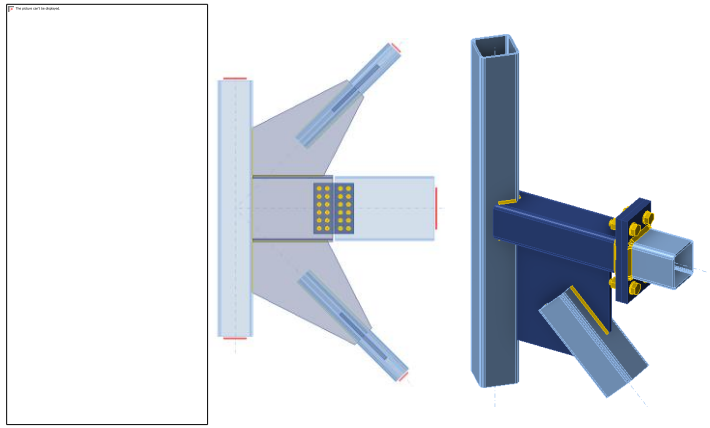


Shear tab connection



Shear tab with reinforced plate

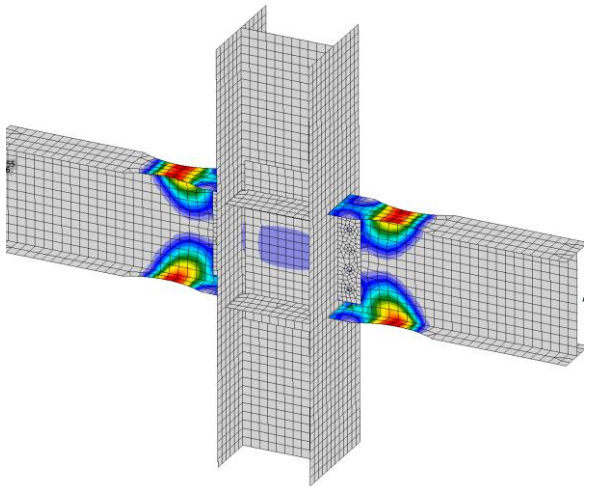
Q&A – NEXT STEPS



More information or questions?

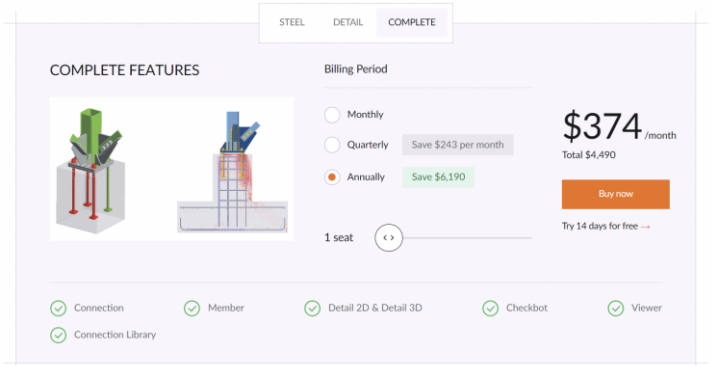
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