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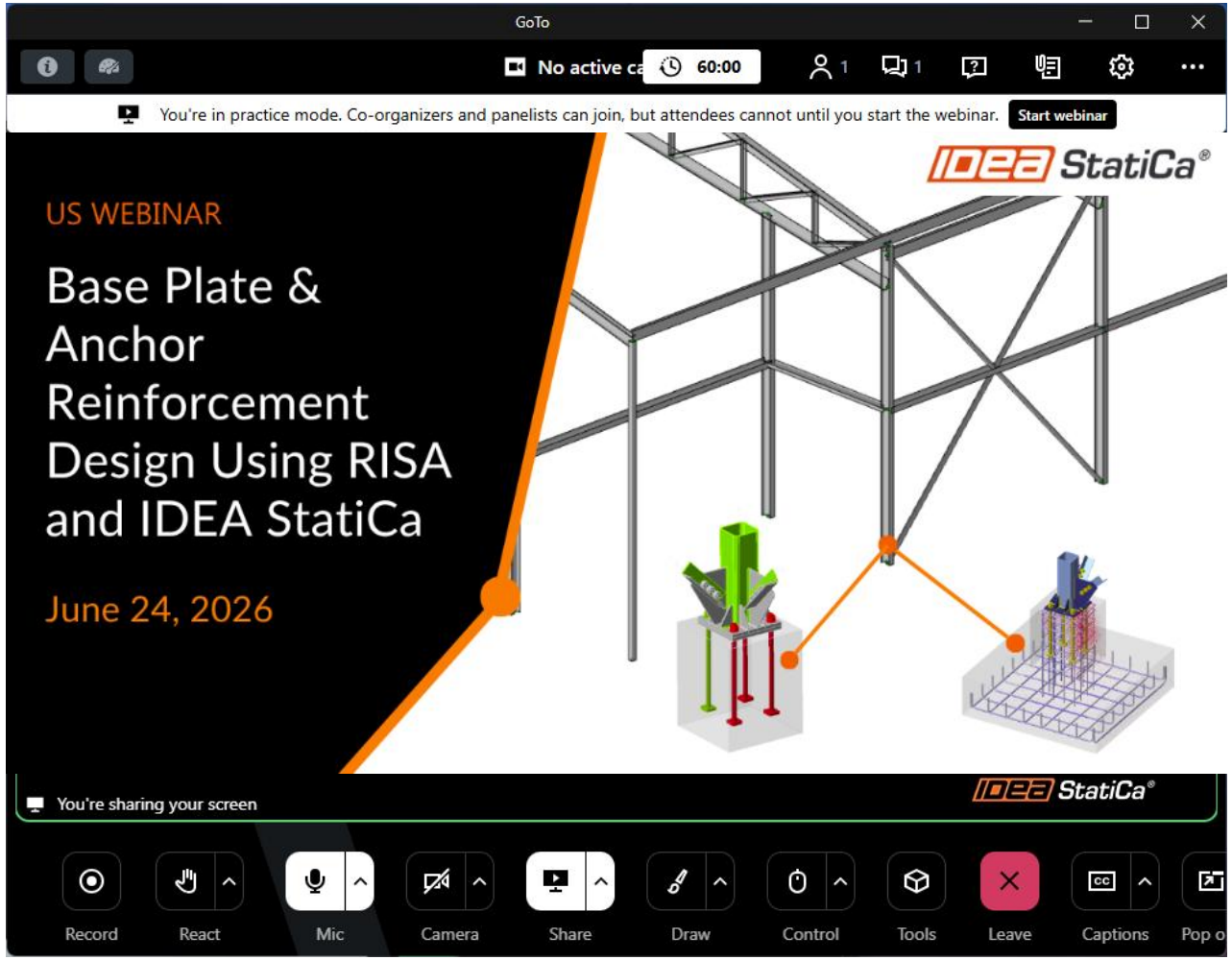
*Calculate yesterday's estimates*

# PRACTICAL GUIDE FOR SEISMIC STEEL CONNECTION DESIGN

# GOTO WEBINAR CONTROL PANEL

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2. To view any shared handouts, select the **Handouts** pane, and then select the desired handout.
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# COMMON INDUSTRIES

- Architectural, High Rise, and Buildings
- Industrial, Plant, & Data Center
- Infrastructure & Transportation
- Construction & Erection







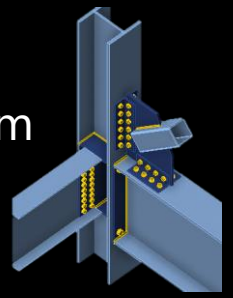


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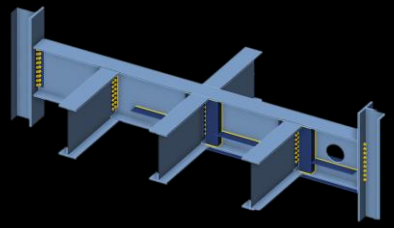
## Connection

Design all steel connections from standard to complex joints



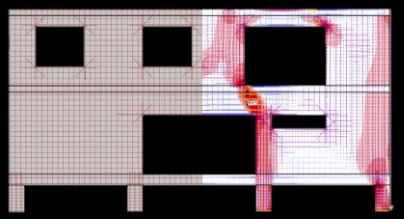
## Member

Critical Beam and column design



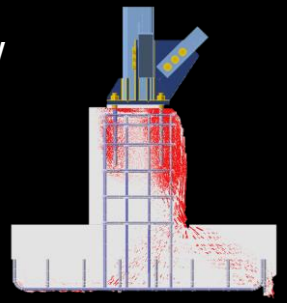
## Detail

Replace STM when designing:
Transfer beams
Shear walls
Corbels
Walking columns



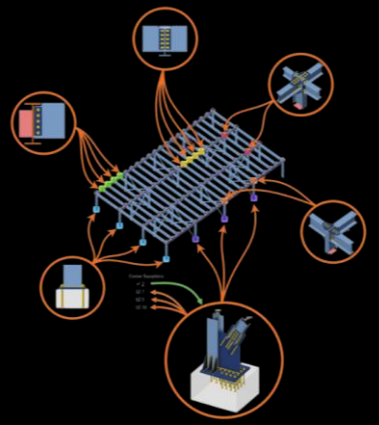
## Connection + Detail

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



## Checkbot

Integration of 3rd party apps analysis models or BIM models with IDEA StatiCa applications



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# AGENDA

- Intro to Capacity design in IDEA StatiCa for seismic connection design

❑ Prequalified steel connection design - Intermediate and special moment frames
❑ Ordinary moment frame Connection design
❑ Braced frames Connection design

- Seismic connection design toolkit

Q&A session

# FRIENDLY REMINDER



This is a summary of the requirements and details for seismic connections, we encourage you to always review the actual standards and examples in the AISC 341 and Seismic Design Manual



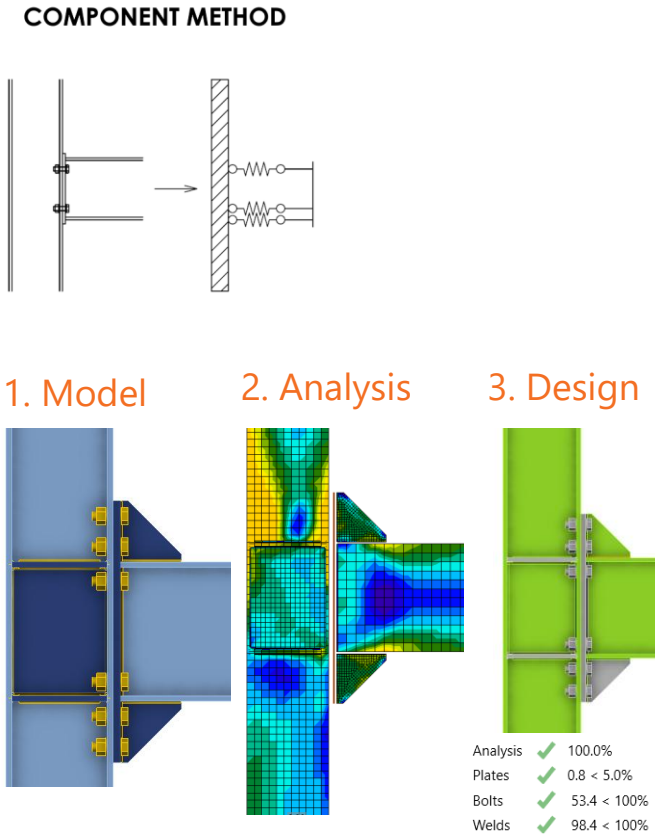
IDEA StatiCa is a tool that should be used in tandem with AISC seismic provisions



Use your engineering judgement and experience

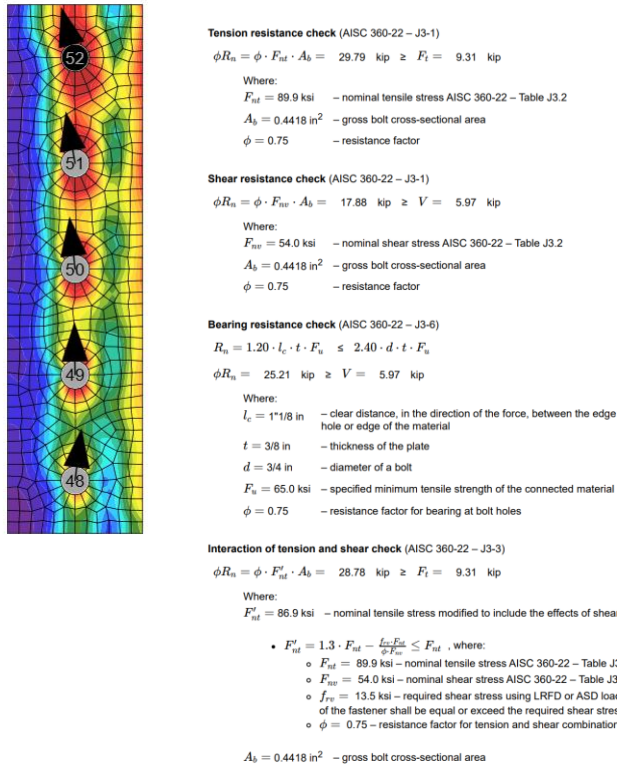
# QUICK INTRO CONNECTION APP

## Component Based Finite Element Method



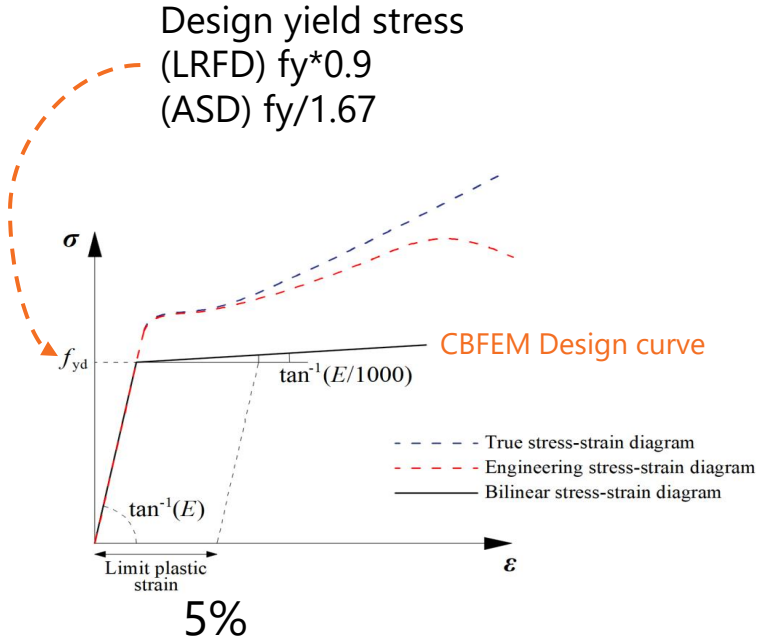

## Code check equations:

✓ Bolts, Welds, anchors




## Steel plates:

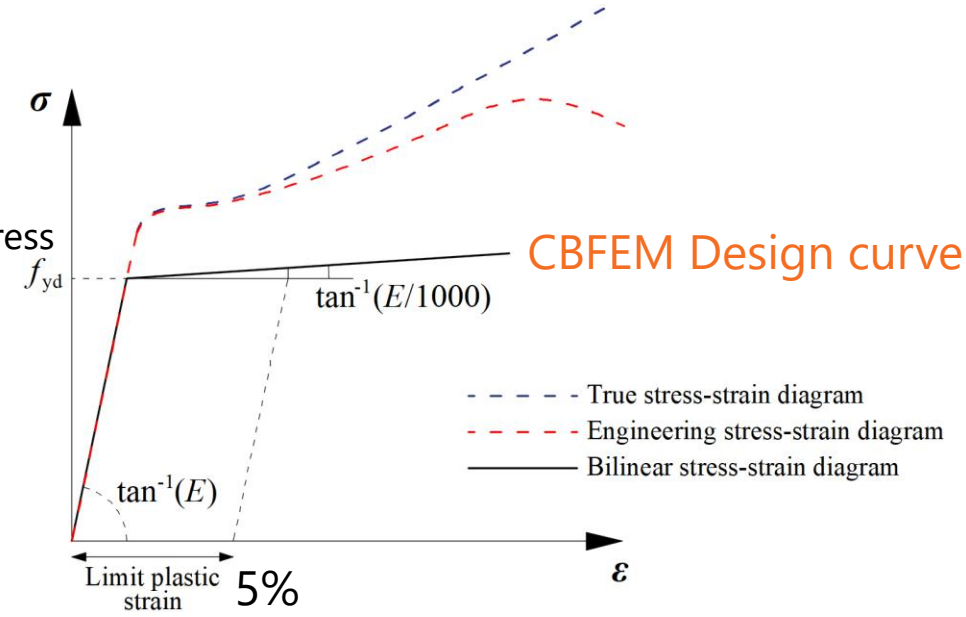
Material properties are checked, design limit is based on plastic strain




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# STEEL MATERIAL MODEL FOR PLATES

Design yield stress
(LRFD) fy*0.9
(ASD) fy/1.67

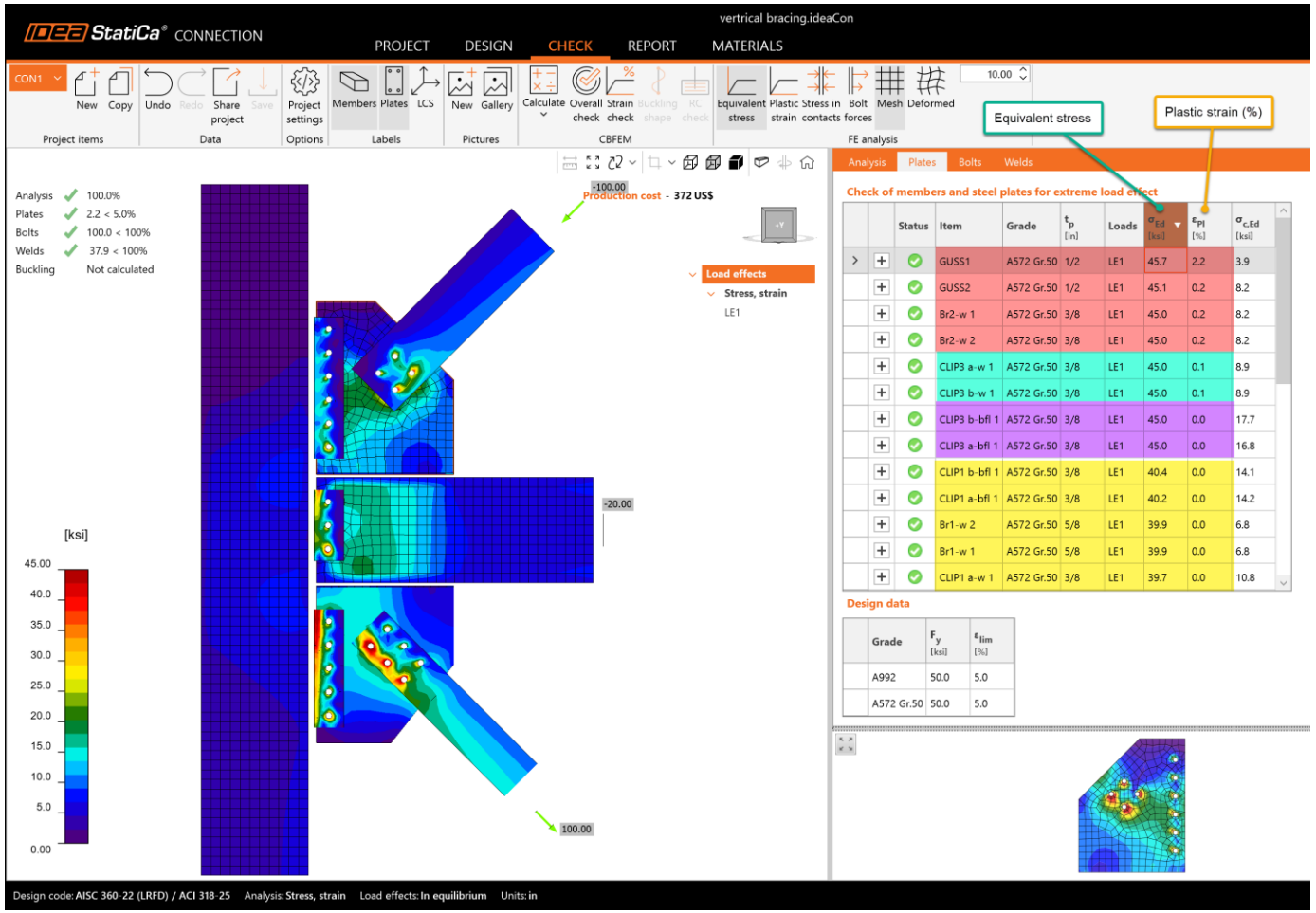



The material behavior is based on the von Mises yield criterion

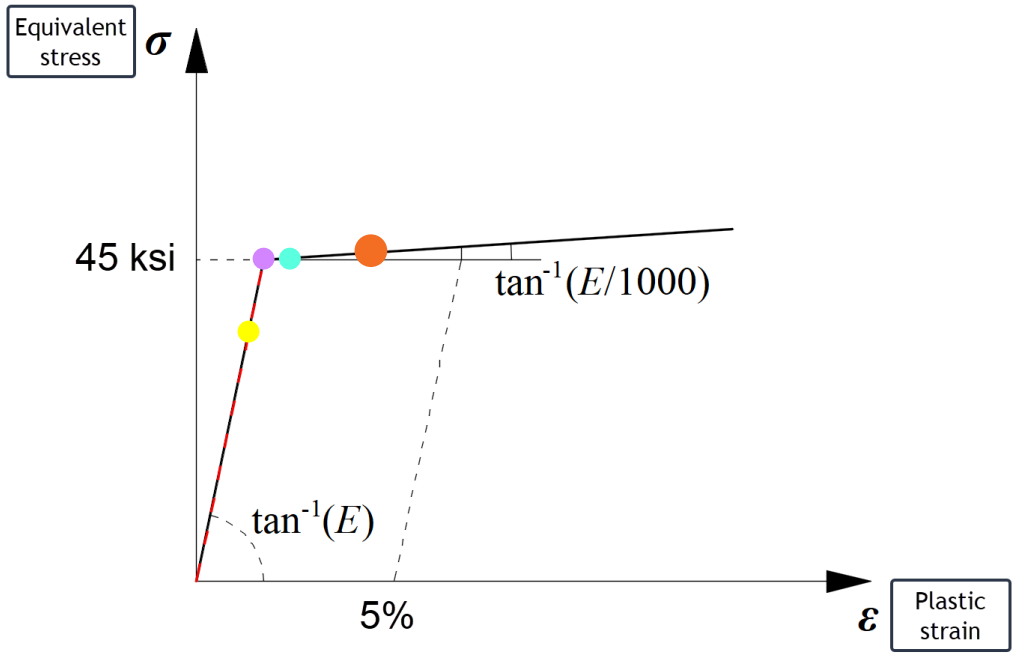
It is assumed to be elastic before reaching the design yield strength, $f_{yd}$.

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# CONNECTION DESIGN (AISC LRFD)



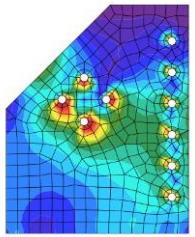

Design yield strength (LRFD) =
50 ksi *0.9 = 45 ksi



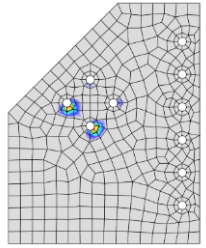

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# CBFEM CODE CHECKS OF STEEL CONNECTION COMPONENTS

**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 | $\sigma_{Ed}$ [ksi] | $\varepsilon_{Pl}$ [%] | $\sigma_{c,Ed}$ [ksi] |
|---|---|---|---|---|---|---|---|---|
| − | ✓ | GUSS1 | A572 Gr.50 | 1/2 | LE1 | 45.7 | 2.2 | 3.9 |

**Design values used in the analysis**

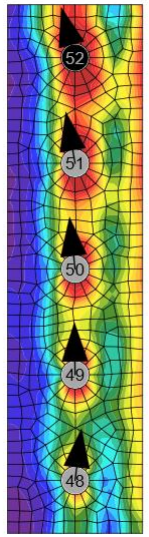
$\phi F_y =$ 45.0 ksi

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




**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

- $F'_{nt} = 1.3 \cdot F_{nt} - \frac{f_{uv}}{\phi F_{nv}} \leq F_{nt}$ , 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
  - $f_{uv} =$ 13.5 ksi – required shear stress using LRFD or ASD loa of the fastener shall be equal or exceed the required shear stre
  - $\phi =$ 0.75 – resistance factor for tension and shear combinatio

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

## Welds

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] | ϕRn [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:

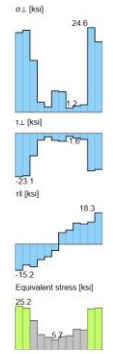
- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot \sin^{1.5}\theta)$ , where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 65.1° – angle of loading measured from the weld longitudinal axis

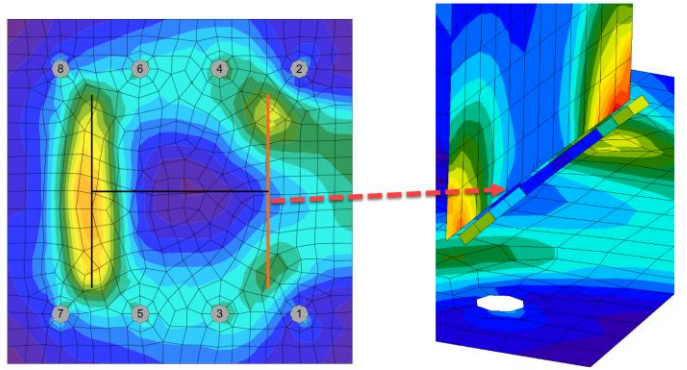
$A_{we} = 0.3732$ in$^2$ – effective area of weld critical element

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

Design data

| Material | $F_{exx}$ [ksi] |
|---|---|
| E70xx | 70.0 |




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# AISC 341 – SEISMIC FORCE-RESISTING SYSTEMS (SFRS)

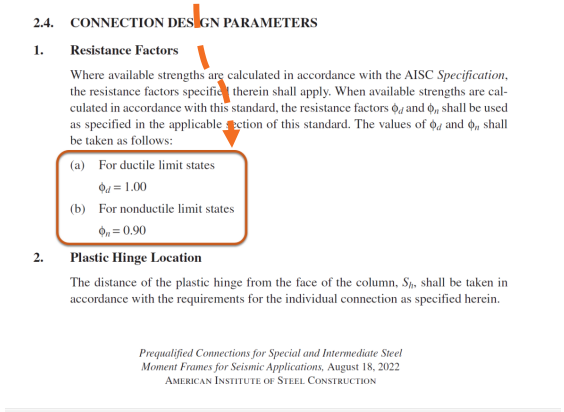
| | |
|---|---|
| Moment frame | • **OMF, IMF, SMF**, STMF, OCCS, SCCS |
| Braced frame and shear walls | • **OCBF**, **SCBF**, EBF, SPSW |
| Composite moment frame | • C-OMF, C-IMF, C-SMF, C-PRMF |
| Composite braced frame and shear wall | • C-OBF, C-SCBF, C-EBF, C-OSW, C-SSW, C-PSW/CE, C-PSW/CF, CC-PSW/CF |

- Ordinary moment frame
- Intermediate moment frame
- Special moment frame
- Ordinary concentrically moment frame
- Special concentrically brace frame

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# AISC 341 SEISMIC CONNECTION DESIGN BASIS

- **Required strength:** The greater from an analysis using seismic effects or **expected strength** from the different system frames chapters
- **Available strength:** Same as specification unless otherwise in AISC 341 or 358



2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

Where available strengths are calculated in accordance with the AISC *Specification*, the resistance factors specified therein shall apply. When available strengths are calculated in accordance with this standard, the resistance factors $\phi_d$ and $\phi_n$ shall be used as specified in the applicable section of this standard. The values of $\phi_d$ and $\phi_n$ shall be taken as follows:

(a) For ductile limit states
$\phi_d = 1.00$

(b) For nonductile limit states
$\phi_n = 0.90$

2. Plastic Hinge Location

The distance of the plastic hinge from the face of the column, $S_h$, shall be taken in accordance with the requirements for the individual connection as specified herein.

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American Institute of Steel Construction



the overstrength seismic load, $E_{mh}$ is permitted to be taken as calculated in ASCE/SEI 7, Equation 12.4-7, with a maximum value of $E_{mh}$ equal to $E_{cl}$.

## B3. DESIGN BASIS

### 1. Required Strength

The required strength of structural members and connections shall be the greater of the following:

(a) The required strength as determined by structural analysis for the applicable load combinations, as stipulated in the applicable building code, and in Chapter C

(b) The required strength given in Chapters D, E, F, G, and H

### 2. Available Strength

The available strength is stipulated as the design strength, $\phi R_n$, for design in accordance with the provisions for load and resistance factor design (LRFD) and the allowable strength, $R_n/\Omega$, for design in accordance with the provisions for allowable strength design (ASD). The available strength of systems, members, and connections shall be determined in accordance with the *Specification*, except as modified throughout these Provisions.

## B4. SYSTEM TYPE

The seismic force-resisting system (SFRS) shall contain one or more moment-frame, braced-frame, or shear-wall system conforming to the requirements of one of the seismic systems designated in Chapters E, F, G, and H.

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# AISC 341 SEISMIC CONNECTION DESIGN BASIS

**6b. FR Moment Connections**

FR moment connections that are part of the seismic force-resisting system (SFRS) shall satisfy one of the following requirements:

(a) FR moment connections shall be designed for a required flexural strength that is equal to the expected beam flexural strength, $R_y M_p$, multiplied by 1.1 and divided by $\alpha_s$, where $\alpha_s$ = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

The required shear strength of the connection, $V_r$, shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$, shall be determined as follows:

$$E_{cl} = 2(1.1R_y M_p)/L_{cf} \qquad \text{(E1-1)}$$

where

$L_{cf}$ = clear length of beam, in. (mm)
$M_p$ = plastic moment, kip-in. (N-mm)
$R_y$ = ratio of expected yield stress to the specified minimum yield stress, $F_y$

**OMF**

indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these Provisions.

**6b. Beam-to-Column Connection Requirements**

Beam-to-column connections used in the SFRS shall satisfy the following requirements:

(a) The connection shall be capable of accommodating a story drift angle of at least 0.02 rad.

(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.02 rad.

**6c. Conformance Demonstration**

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E2.6b by one of the following:

(a) Use of IMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for IMF in accordance with Section K1.





(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2

**6d. Required Shear Strength**

The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$, shall be determined as follows:

**IMF**





**6b. Beam-to-Column Connections**

Beam-to-column connections used in the seismic force-resisting system (SFRS) shall satisfy the following requirements:

(a) The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.

(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.04 rad, unless equivalent performance of the moment frame system is demonstrated through substantiating analysis conforming to ASCE/SEI 7, Sections 12.2.1.1 or 12.2.1.2,

where

$M_p$ = plastic moment, kip-in. (N-mm)

**6c. Conformance Demonstration**

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E3.6b by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Section K1.

(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:

(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2

**6d. Required Shear Strength**

The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$, shall be taken as follows:

$$E_{cl} = 2M_{pr}/L_h \qquad \text{(E3-5)}$$

**SMF**

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# PROBABLE MAXIMUM MOMENT AT PLASTIC HINGE

AISC 358





**3. Probable Maximum Moment at Plastic Hinge**

The probable maximum moment at the plastic hinge shall be

$$M_{pr} = C_{pr} R_y F_y Z_e \quad (2.4\text{-}1)$$

where

$C_{pr}$ = factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Unless otherwise specifically indicated in this standard, the value of $C_{pr}$ shall be

$$C_{pr} = \frac{F_y + F_u}{2F_y} \le 1.2 \quad (2.4\text{-}2)$$

$F_u$ = specified minimum tensile strength of yielding element, ksi (MPa)
$F_y$ = specified minimum yield stress of yielding element, ksi (MPa)
$R_y$ = ratio of the expected yield stress to the specified minimum yield stress, $F_y$, as specified in the AISC *Seismic Provisions*
$Z_e$ = effective plastic section modulus of section (or connection) at location of the plastic hinge, in.$^3$ (mm$^3$)

**4. Shear Force at Plastic Hinge**

Unless otherwise specifically indicated in this standard, the shear force at the plastic hinge, $V_h$, shall be

$$V_h = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (2.4\text{-}3)$$

where

$L_h$ = distance between plastic hinge locations, in. (mm)
$V_{gravity}$ = shear force resulting from $1.2D + f_1 L + 0.15S$ acting on the beam between plastic hinge locations, where $D$ is the dead load, $S$ is the snow load, and $f_1$ is the load factor determined by the applicable building code for live loads, but not less than 0.5, kips (N)

In applying Equation 2.4-3, the values of $2M_{pr}/L_h$ and $V_{gravity}$ shall be taken as having the same sign.

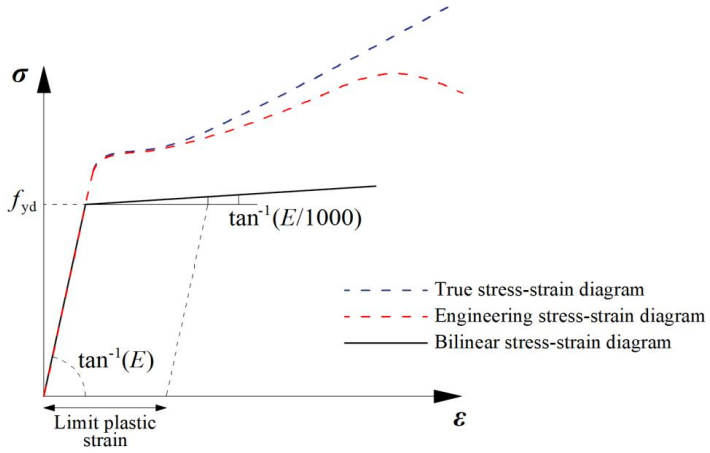
**User Note:** The load combination of $1.2D + f_1 L + 0.15S$ is in conformance with ASCE/SEI 7-22. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for $S$ when the roof configuration is such that it does not shed snow off of the structure.

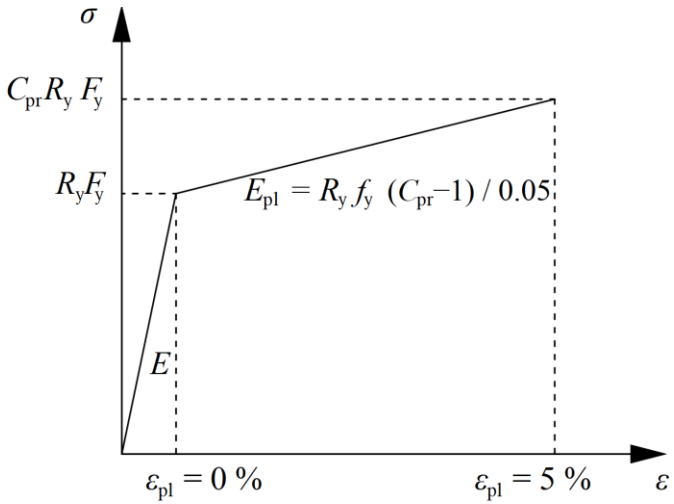
**User Note:** Equation 2.4-3 can be derived from a free-body diagram of the portion of the beam between plastic hinge locations, assuming the moment at each plastic hinge is $M_{pr}$ and with gravity load acting on the free-body diagram. The

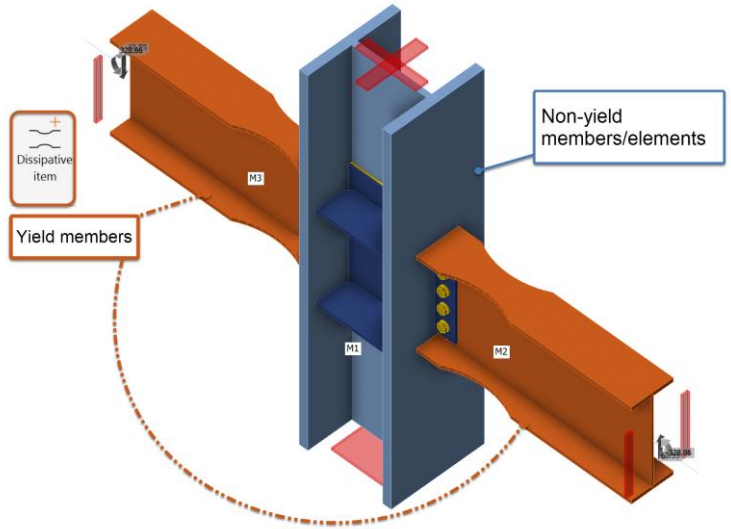
IDEA StatiCa®

# SEISMIC DESIGN IN IDEA STATICA

## CAPACITY DESIGN




## Non-yield member design material curve

fyd affected by material factors

LRFD= Fy*0.9

ASD= Fy/1.67

**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

## Yield member design material curve

Ry= Overstrenght factor

Cpr*=Hardening factor

**Design data**

| | Grade | ϕ [-] | Ry [-] | Fy [ksi] | Fy,FEM [ksi] | Cpr [-] | ε,lim [%] |
|---|---|---|---|---|---|---|---|
| › | A992 | 0.90 | - | 50.0 | 45.0 | - | 5.0 |
| | A992 * | - | 1.10 | 50.0 | 55.0 | 1.15 | 5.0 |

## Selected dissipative members aren't checked

Column and connections are checked using design limits

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# DISSIPATIVE ITEM SELECTION (YIELD MEMBER)

OMF, IMF, SMF = Beam is the yield member

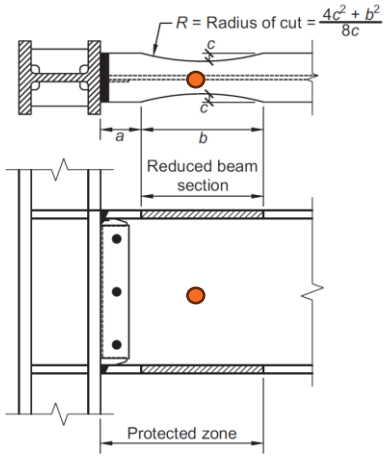



*Fig. 5.1. Reduced beam section connection.*

*Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, August 18, 2022
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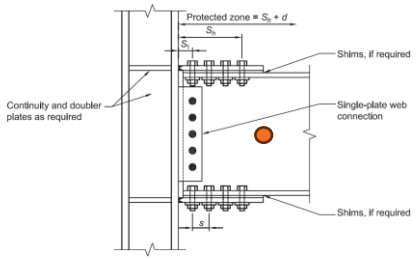



*Fig. 7.1. Bolted flange plate moment connection.*

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Sect. 13.1.] GENERAL 9.2-137

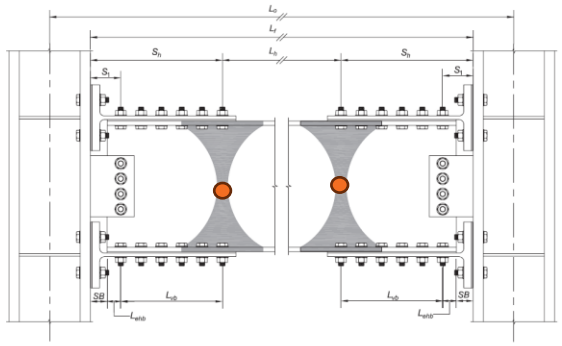
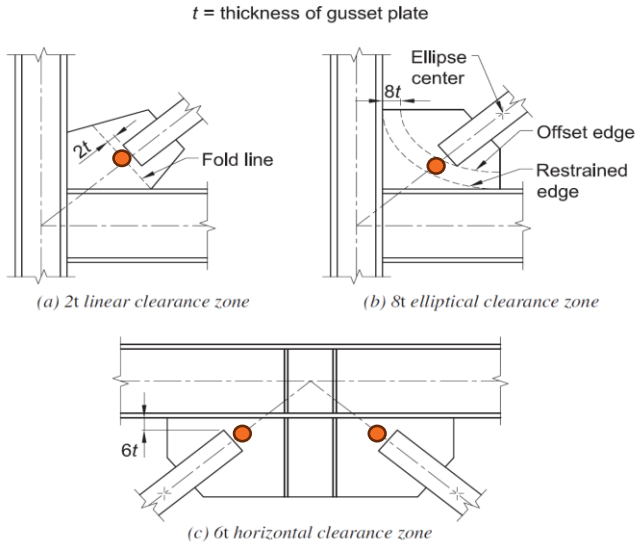



*Fig. 13.2. Beam dimensions for double-tee connections. Shaded regions represent plastic hinges.*

OCBF, SCBF= Brace is the yield member




*(a) 2t linear clearance zone* *(b) 8t elliptical clearance zone*

*(c) 6t horizontal clearance zone*

*Fig. C-F2.18. Brace-to-gusset plate requirement for buckling out-of-plane bracing system.*

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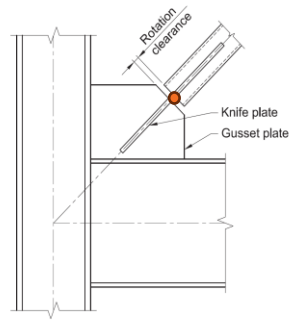


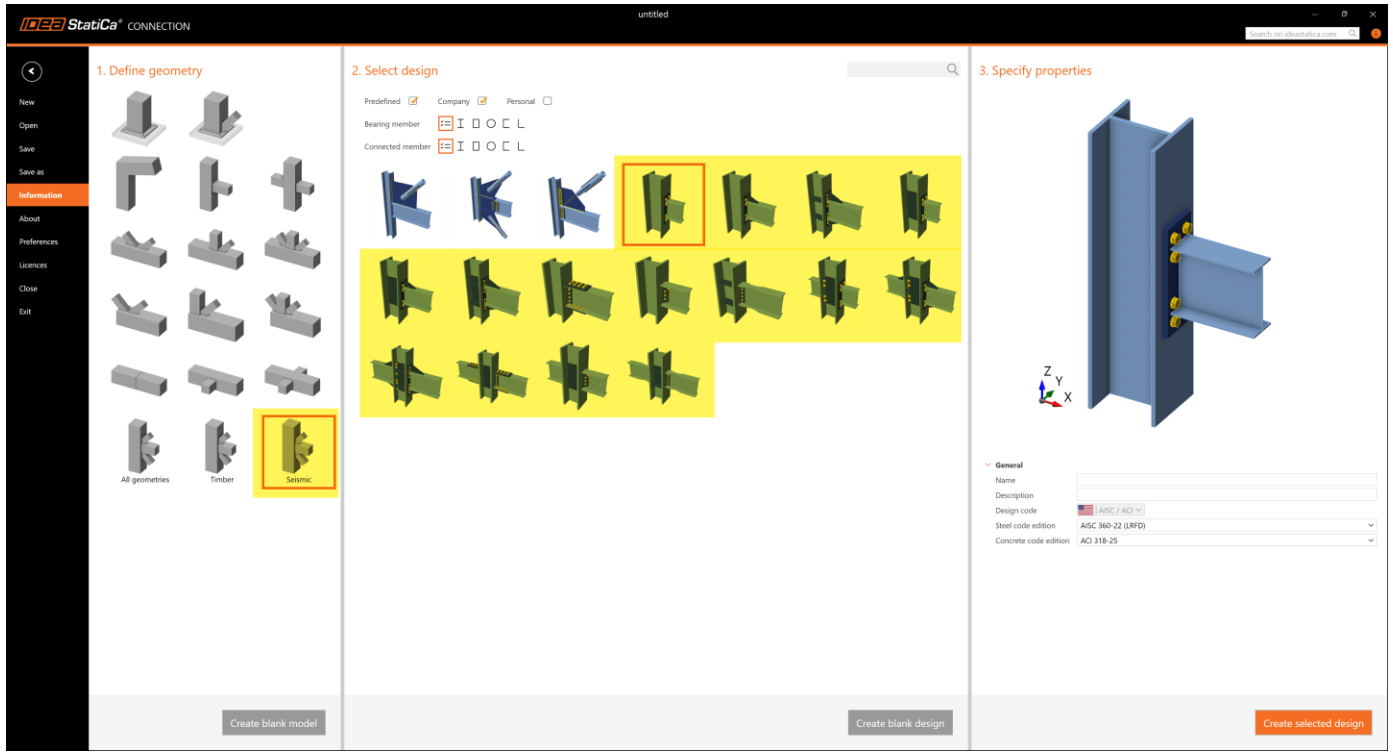

*Fig. C-F2.20. Gusset designed for in-plane rotation (Tsai et al., 2013).*

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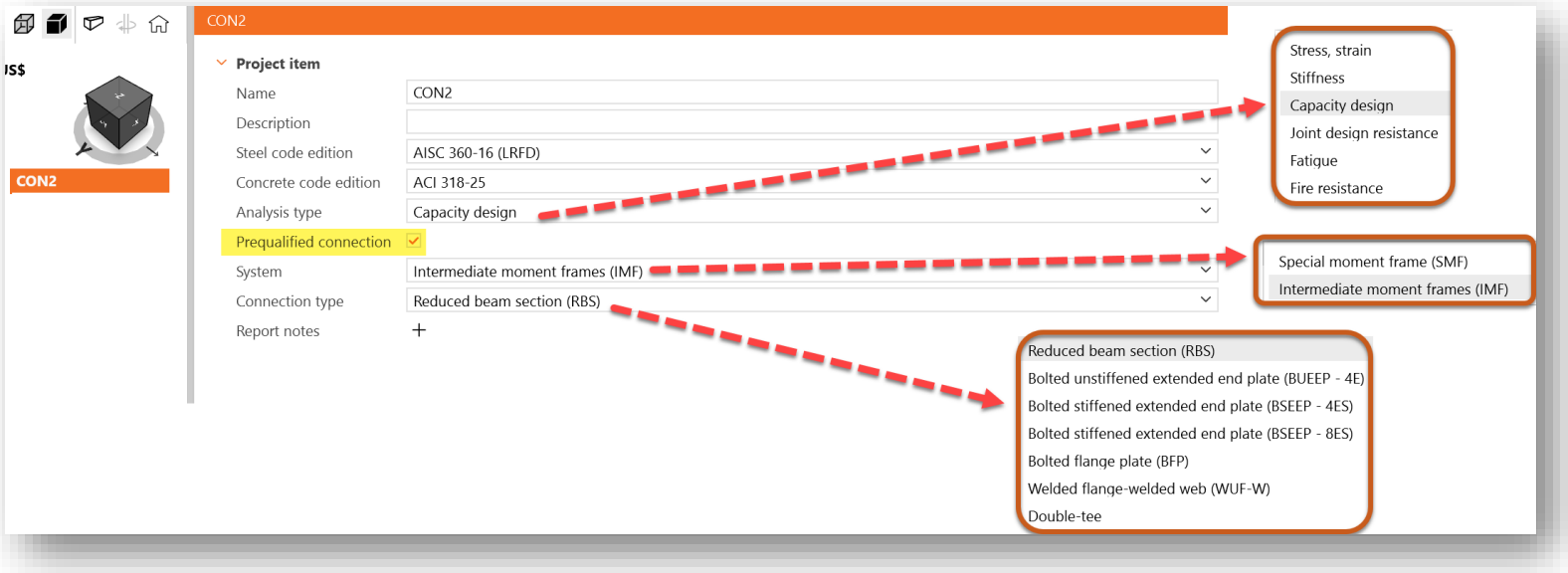
# PREQUALIFIED CONNECTIONS TEMPLATES

1. Reduced Beam Section (RBS)
2. Bolted Unstiffened and Stiffened Extended End-plate
3. Bolted Flange Plate (BFP)
4. Welded Unreinforced Flange-Welded Web (WUF-W)
5. Double-Tee



# PROJECT ANALYSIS TYPE

- ✓ Check the prequalified connection box to get detailing checks
- ✓ Select the appropriate frame system
- ✓ Select the correct connection type



# EXAMPLE 4.3.5. SMF BEAM-COLUMN CONNECTION DESIGN—RBS

## Example 4.3.5. SMF Beam-Column Connection Design—RBS

The SMF beam-column connection design presented in this example has been chosen to demonstrate the application of the design provisions for prequalified RBS connections in accordance with ANSI/AISC 358. This example demonstrates that the RBS geometry given in Figure 4-11, repeated here, is satisfactory. Some of the results from this example are

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used in Example 4.3.3. The geometry of an RBS connection is not unique, and alternative configurations of the RBS geometry are possible.

**Given:**

Refer to Joint JT-1 in Figure 4-10, repeated here. Design the connection between Beam BM-1 and Column CL-1 using the RBS shown in Figure 4-11. All beams and columns are ASTM A992/A992M W-shapes. Use ASTM A572/A572M Grade 50 plate material and 70 ksi electrodes. The gravity loads on the beam are as follows:

$w_D = 0.840$ kip/ft $\quad\quad$ $w_L = 0.600$ kip/ft

**Procedure:**

The procedure outlined in this example follows the design procedure outlined in ANSI/AISC 358, Section 5.7. The term "Step *n*" indicates the actual step number in ANSI/AISC 358, Section 5.7. The steps from ANSI/AISC 358 are augmented with some additional checks in this example. Some of the steps listed in Table 4-A are executed in detail in Example 4.3.3, the SMF beam strength check. Because ANSI/AISC 358 addresses LRFD only, the following procedure is also only defined for LRFD.

In addition, column bracing requirements are checked and the beam flange-to-column flange connection is specified.



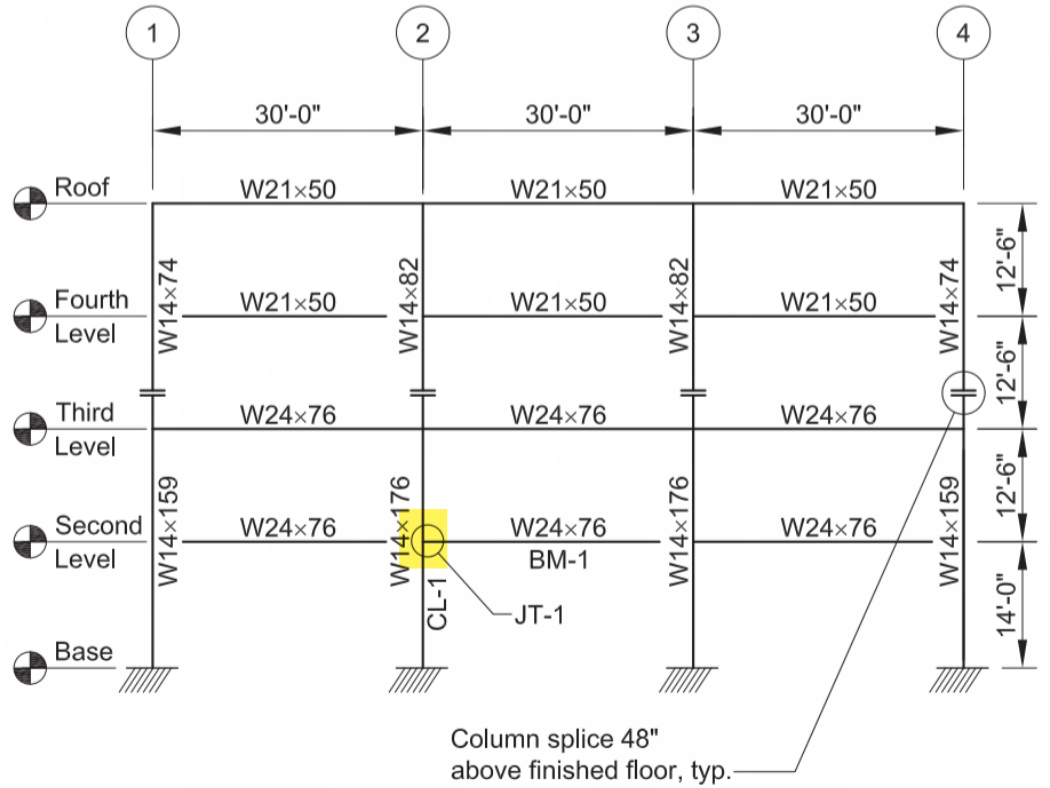



*Fig. 4-10. SMF elevation.*
*(Repeated here for convenience)*

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# FINAL DESIGN MODEL

Design example 4.3.5 – RBS Example



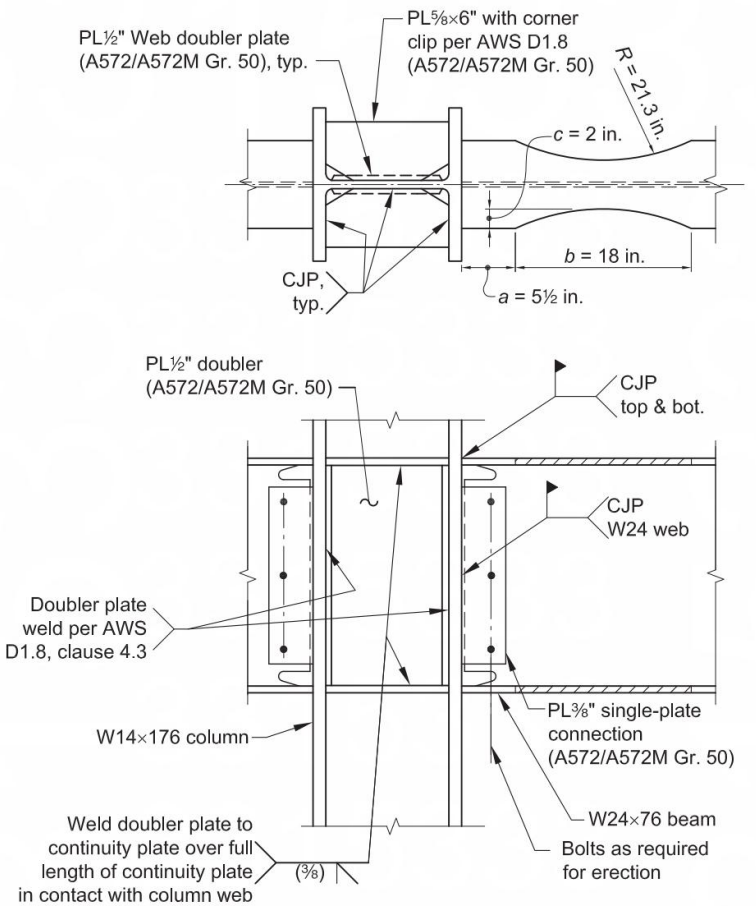
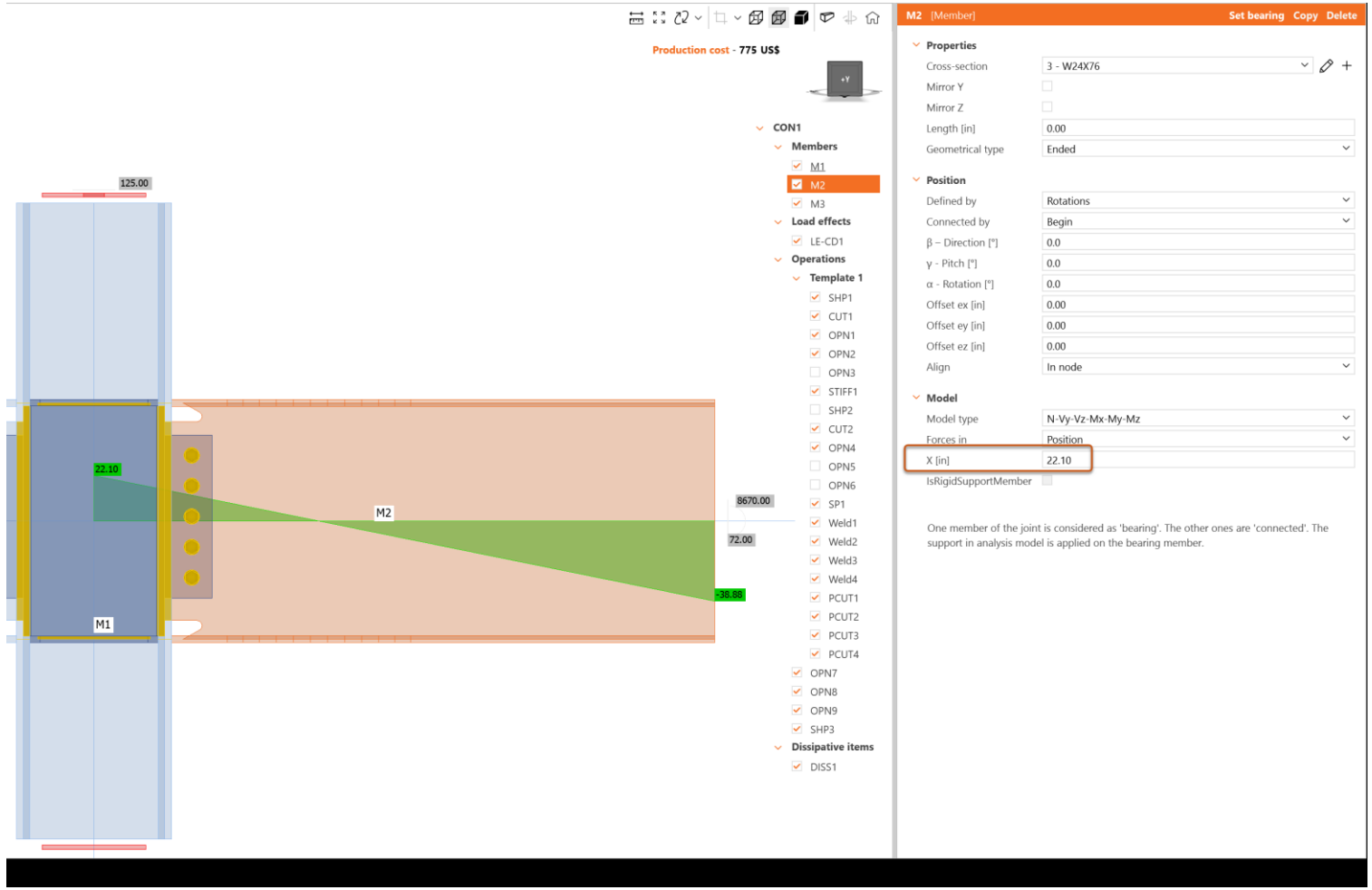


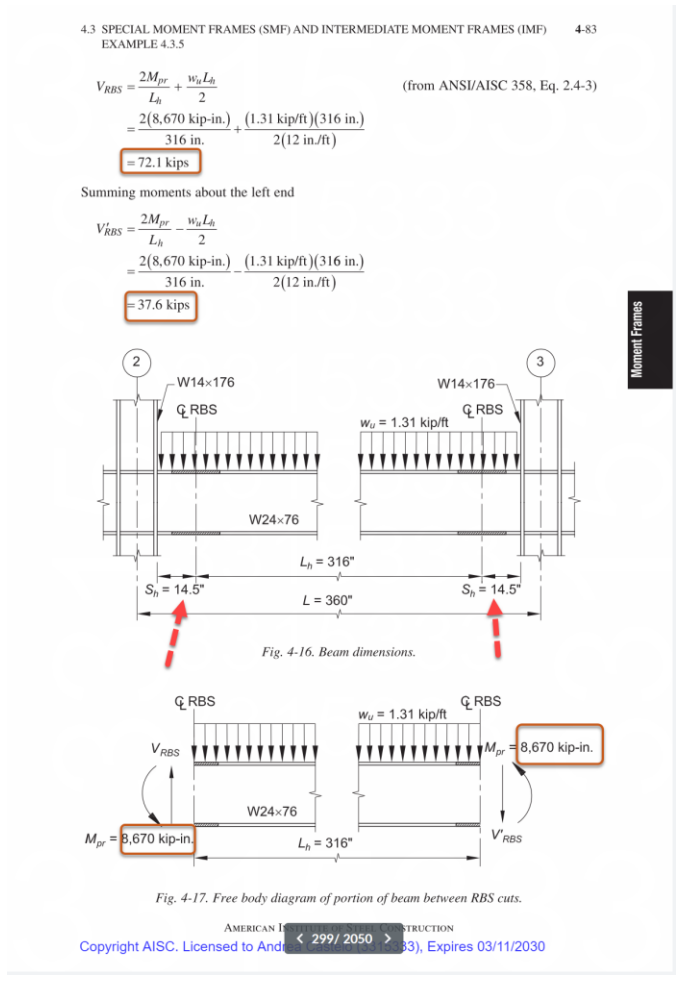

*Fig. 4-20 (continued). Design Example 4.3.5 connection geometry.*

# DESIGN PROCESS IN IDEA STATICA (MOMENT FRAMES)

1. Model the geometry and connection, design it for the regular load combinations (non seismic) using stress/strain analysis in IDEA StatiCa.
2. Copy the connection and change the analysis type to capacity design (select prequalified connection box, system and connection type)
3. Select the beams as dissipative items
4. Calculate the probable maximum moment at the plastic hinge, determine the distance of the plastic hinge, and moment due to the shear force coming from top and bottom column
5. Input the forces in IDEA StatiCa, decide to locate it at the plastic hinge location or column face. Add shear force in the column to input the moment at the node
6. If ductile behavior is required, modify the design factors for the non-yield members and connection elements
7. Run the analysis and verify the maximum plastic strain happens at the planned hinge in the beams. Verify all the connecting elements and column are complying with the design checks and plastic strain less than 5%.

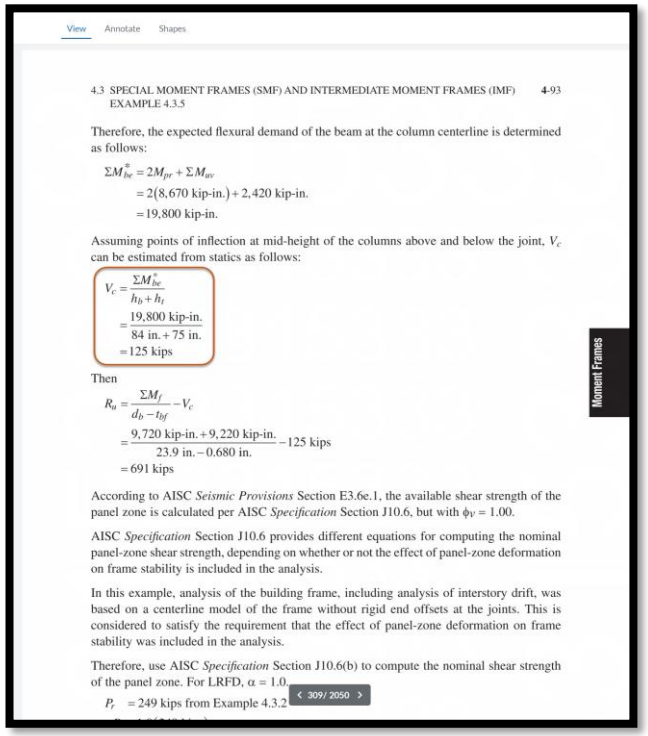
# FORCE LOCATION (RBS EXAMPLE)



# BALANCE LOADS USING THE COLUMN (RBS EX)

Moment= 159 in x 125 kips = **19875 kip-in**



4.3 SPECIAL MOMENT FRAMES (SMF) AND INTERMEDIATE MOMENT FRAMES (IMF) 4-93
EXAMPLE 4.3.5

Therefore, the expected flexural demand of the beam at the column centerline is determined as follows:

$$\Sigma M_{be}^{*} = 2M_{pr} + \Sigma M_{av}$$
$$= 2(8{,}670 \text{ kip-in.}) + 2{,}420 \text{ kip-in.}$$
$$= 19{,}800 \text{ kip-in.}$$

Assuming points of inflection at mid-height of the columns above and below the joint, $V_c$ can be estimated from statics as follows:

$$V_c = \frac{\Sigma M_{be}^{*}}{h_b + h_t} = \frac{19{,}800 \text{ kip-in.}}{84 \text{ in.} + 75 \text{ in.}} = 125 \text{ kips}$$

Then

$$R_u = \frac{\Sigma M_f}{d_b - t_{bf}} - V_c = \frac{9{,}720 \text{ kip-in.} + 9{,}220 \text{ kip-in.}}{23.9 \text{ in.} - 0.680 \text{ in.}} - 125 \text{ kips} = 691 \text{ kips}$$

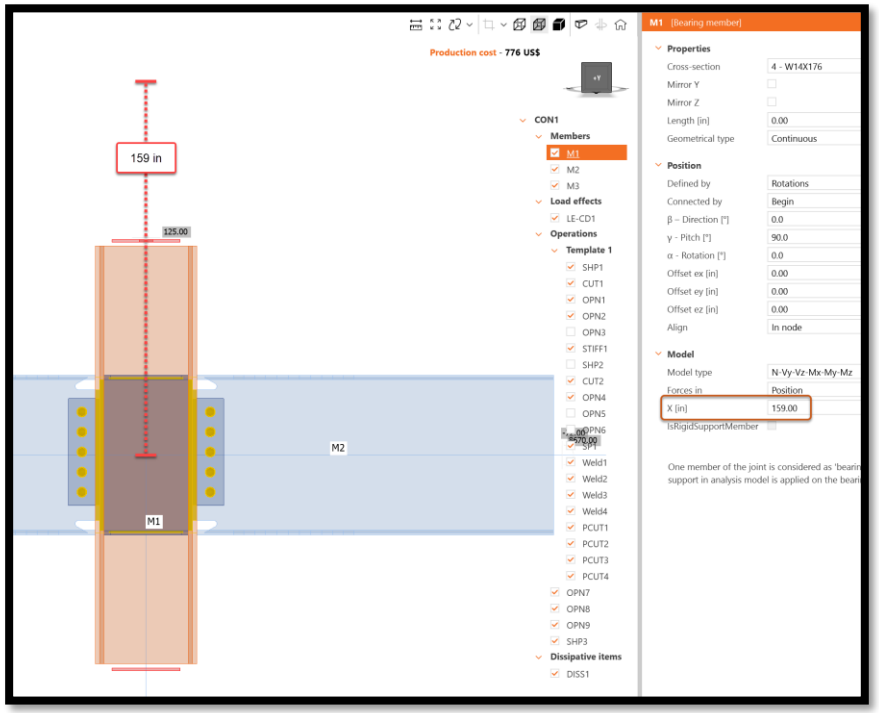
According to AISC *Seismic Provisions* Section E3.6e.1, the available shear strength of the panel zone is calculated per AISC *Specification* Section J10.6, but with $\phi_v = 1.00$.

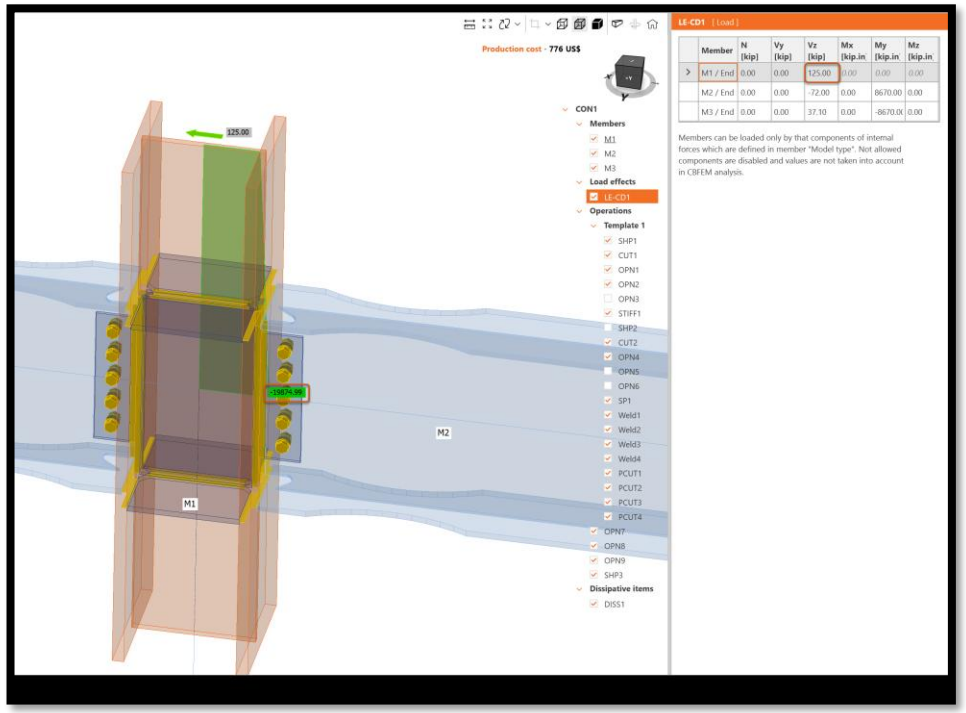
AISC *Specification* Section J10.6 provides different equations for computing the nominal panel-zone shear strength, depending on whether or not the effect of panel-zone deformation on frame stability is included in the analysis.

In this example, analysis of the building frame, including analysis of interstory drift, was based on a centerline model of the frame without rigid end offsets at the joints. This is considered to satisfy the requirement that the effect of panel-zone deformation on frame stability was included in the analysis.

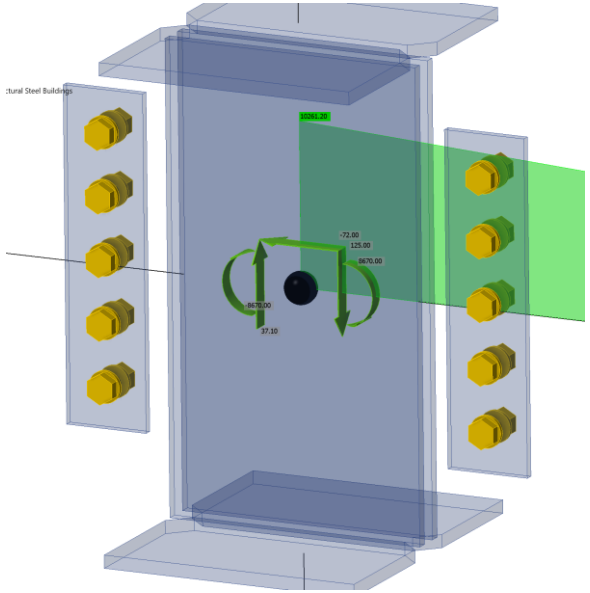
Therefore, use AISC *Specification* Section J10.6(b) to compute the nominal shear strength of the panel zone. For LRFD, $\alpha = 1.0$.

$P_r$ = 249 kips from Example 4.3.2

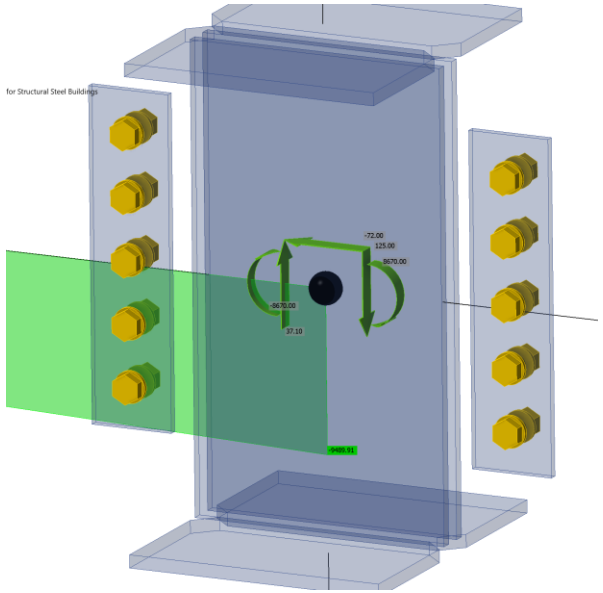


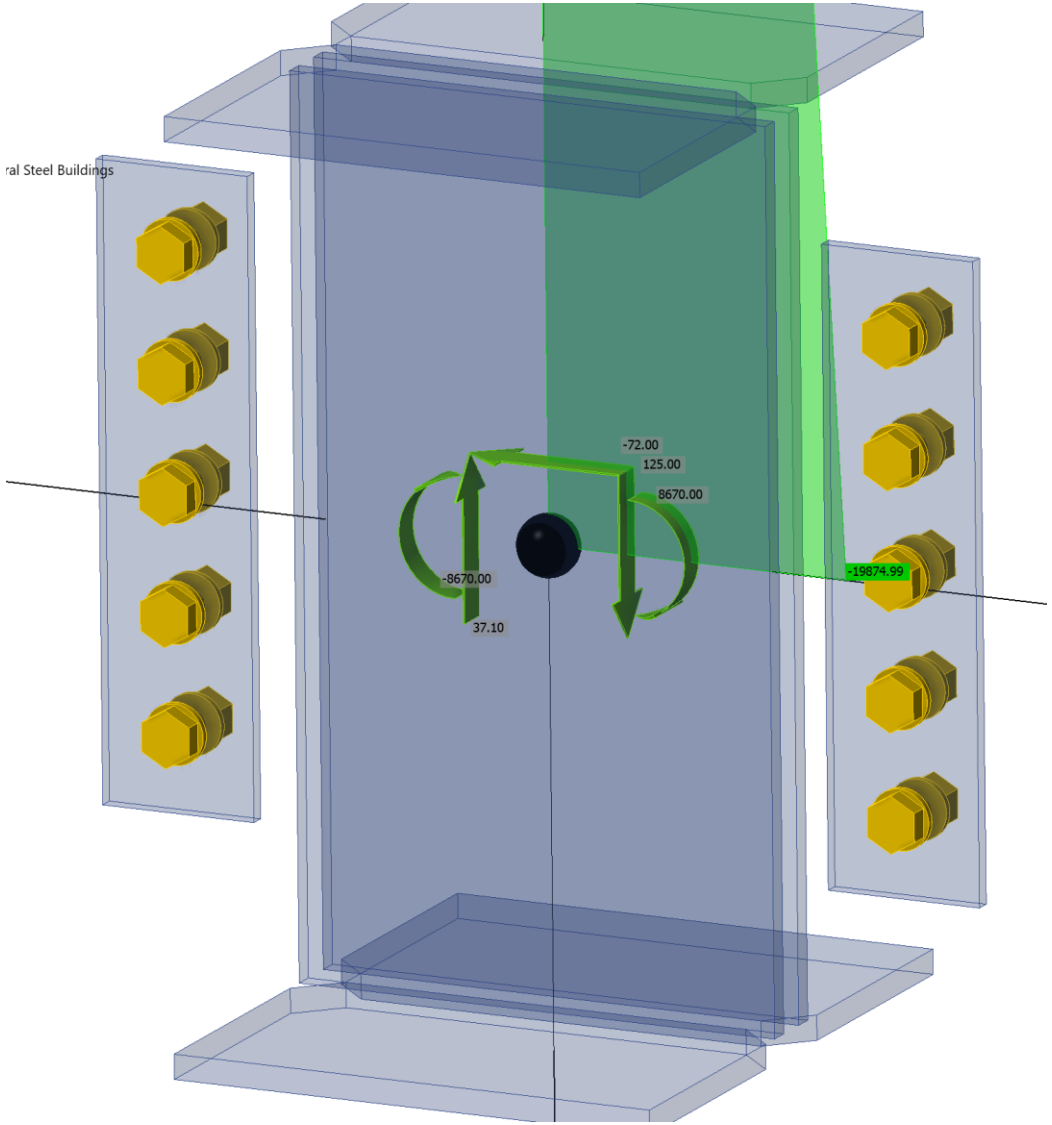
Moment values and diagram at the node



Right beam



Left beam




Column

# RESISTANCE FACTORS FOR AISC 358

## 2.4. CONNECTION DESIGN PARAMETERS

### 1. Resistance Factors

Where available strengths are calculated in accordance with the AISC *Specification*, the resistance factors specified therein shall apply. When available strengths are calculated in accordance with this standard, the resistance factors $\phi_d$ and $\phi_n$ shall be used as specified in the applicable section of this standard. The values of $\phi_d$ and $\phi_n$ shall be taken as follows:

(a) For ductile limit states

$\phi_d = 1.00$

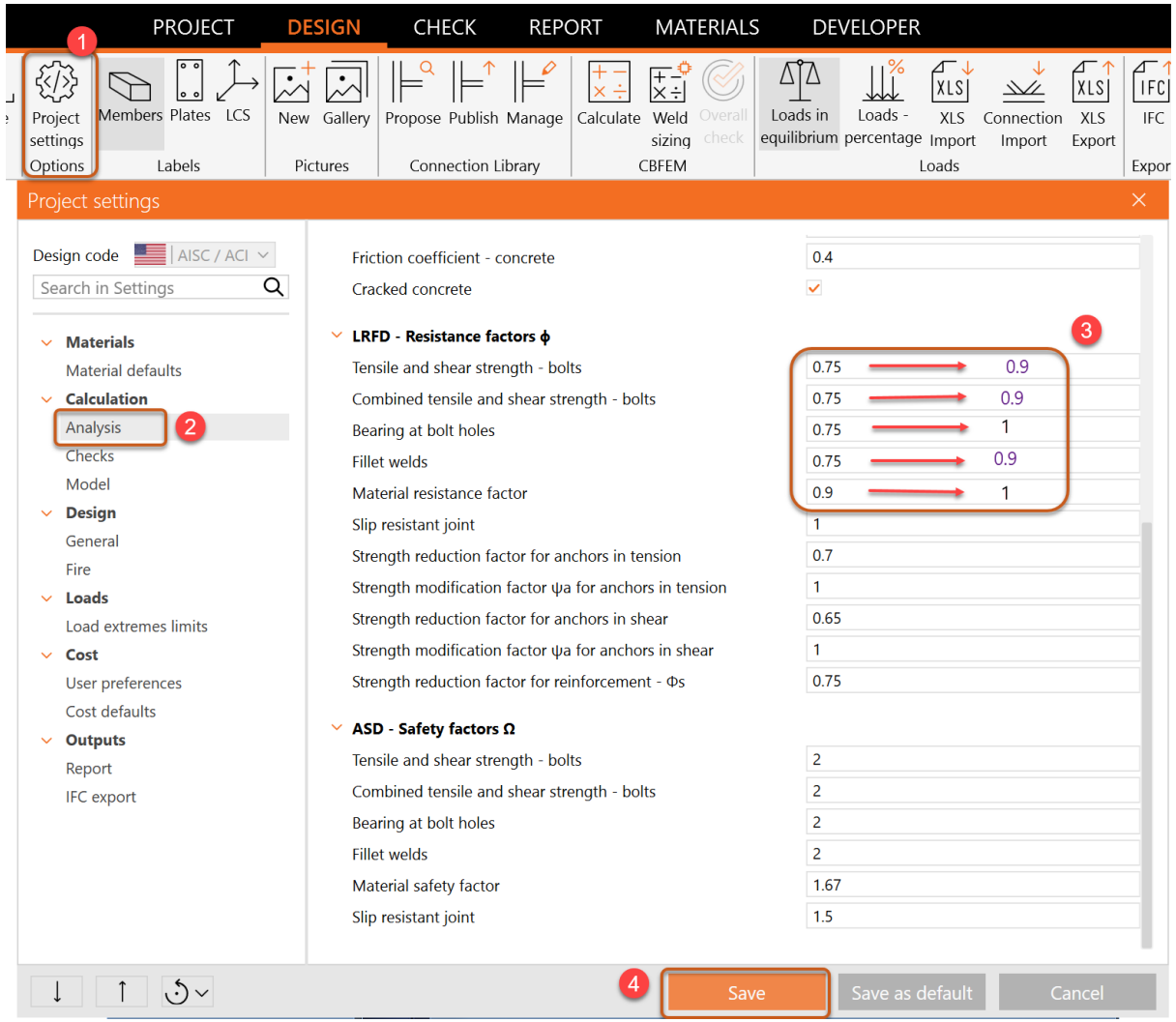
(b) For nonductile limit states

$\phi_n = 0.90$

### 2. Plastic Hinge Location

The distance of the plastic hinge from the face of the column, $S_h$, shall be taken in accordance with the requirements for the individual connection as specified herein.

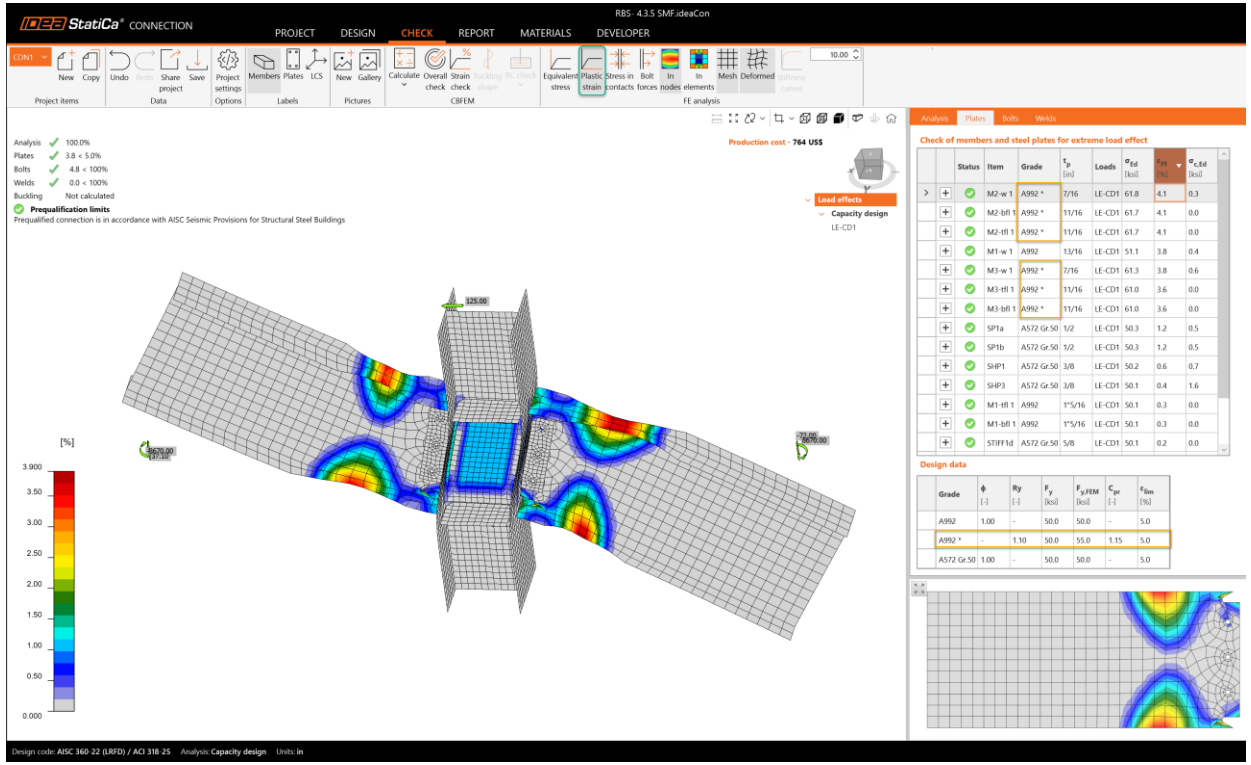
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# OUTCOME

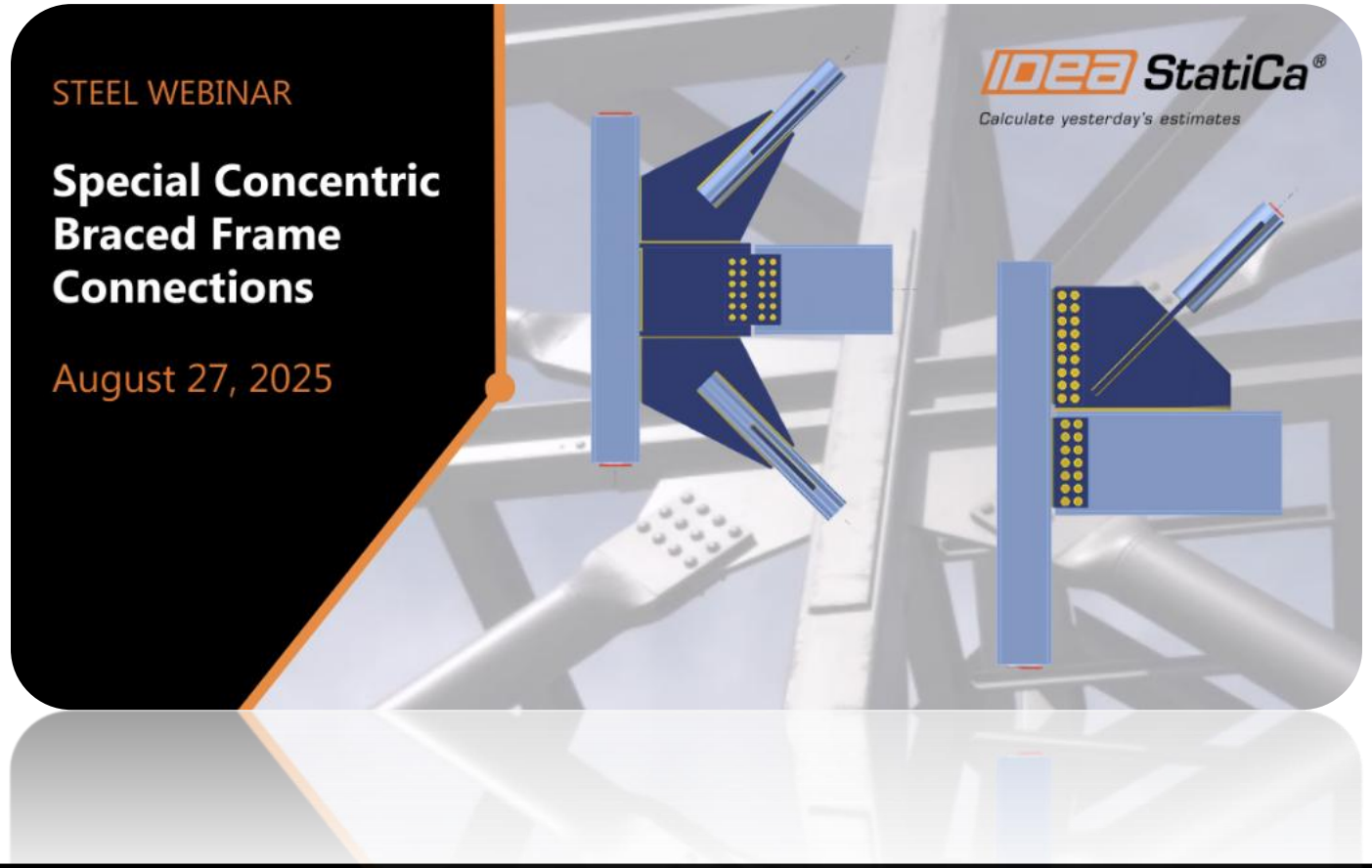
- ✓ Detailing checks for RBS SMF Model
- ✓ Plastic hinge confirmation and visualization
- ✓ Connection design
- ✓ Complete report with checks and detailing drawings/sketches
- ✓ See attached report



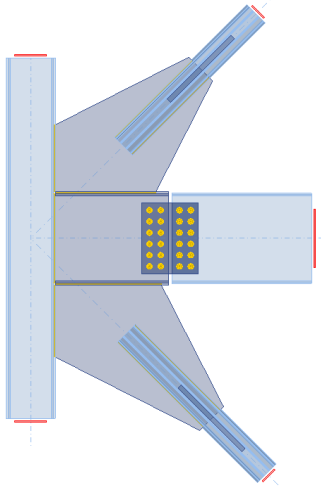

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# WEBINAR (2025): SPECIAL CONCENTRIC BRACED FRAME CONNECTIONS

- ✓ AISC 341 braced frame systems and connections
- ✓ SCBF connection design in IDEA StatiCa

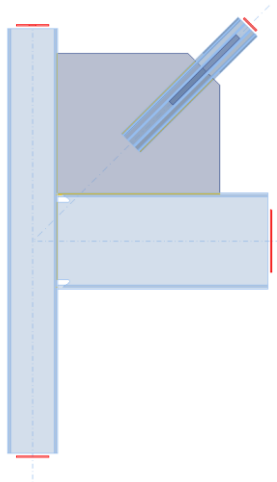


# EXAMPLES – AISC SEISMIC DESIGN MANUAL 4TH ED.

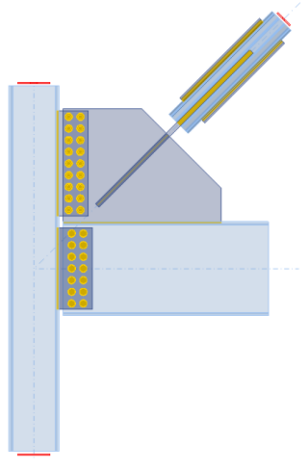


5.3.10 SCBF Brace-to-Beam/Column Connection Design

Modeling from scratch



5.3.11 SCBF Brace-to-Beam/Column Connection Design with Elliptical Clearance and Fixed Beam-to-Column Connection



5.3.12 SCBF Brace-to-Beam/Column Connection Design—In-Plane Brace Buckling

Modeling from scratch

# EXAMPLES – AISC SEISMIC DESIGN MANUAL 4TH ED.

## LIVE DEMO

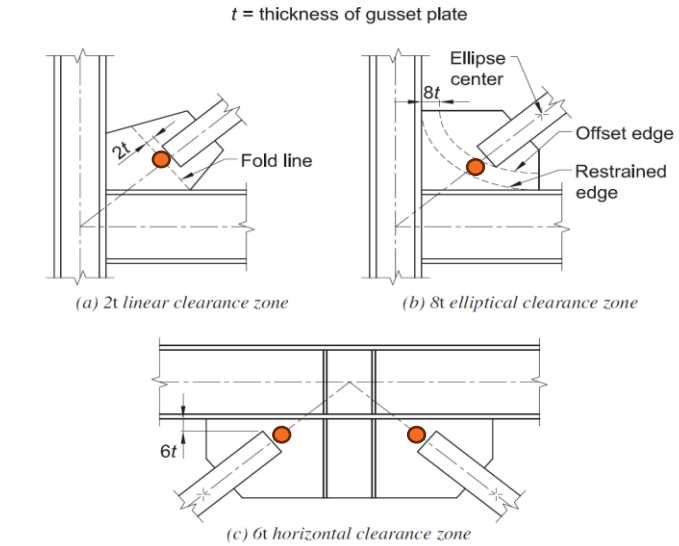
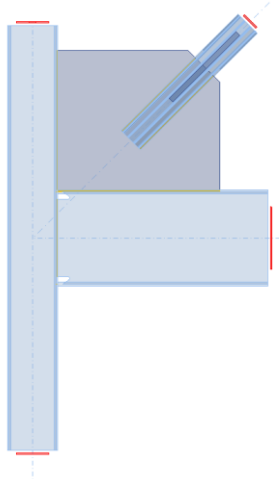



*Fig. C-F2.18. Brace-to-gusset plate requirement for buckling out-of-plane bracing system.*

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5.3.11 SCBF Brace-to-Beam/Column Connection Design with Elliptical Clearance and Fixed Beam-to-Column Connection

# DESIGN PROCESS FOR EXAMPLE 5.3.11

1. **Select** cross sections for bracing members – AISC
2. Calculate **required strength** for the **bracing members** (from the seismic analysis and capacity-limited strength) – AISC
3. Calculate the initial **gusset geometry** ( gusset plate basic geometry depending on the method to use linear hinge, elliptical hinge, out of plane bracing) – AISC
4. Calculate the initial gusset plate **thickness** using the code formula as starting point – AISC
5. Select the type of connection for the beam to column (simple, fully restrained, OMF) – AISC
6. Set up the **model** in IDEA StatiCa
7. Design (stress/strain) the beam to column connection using gravity loads and run a **stiffness analysis** to verify the classification (simple or fully restrained)
8. Select **Capacity design**, Input the loads for the bracing from point 1 and balance them, select the dissipative members.
9. Run 1st iteration – **check the connection** for instance: member failing for the tension case, weld brace to gusset
10. Run 2nd 3, 4 … iterations – Modify the design as needed and run until the connection **passes all the checks**
11. Finally run a **buckling analysis** to prove the buckling happens in the gusset plate

| AISC 341 | IDEA StatiCa |
| --- | --- |
| Design connection elements for expected strength | Capacity design |
| Design beam to column connection to accommodate demands corresponding to large drifts | Stiffness analysis |
| Gusset design: Linear and elliptical hinge zone, hinge plate | Advanced operations to model special gusset geometry, parametric design |
| Design the brace end condition to maintain its integrity as the brace buckles | Buckling analysis |

# AISC 341

# IDEA StatiCa

Design connection elements for expected strength

Capacity design

Design beam to column connection to accommodate demands corresponding to large drifts

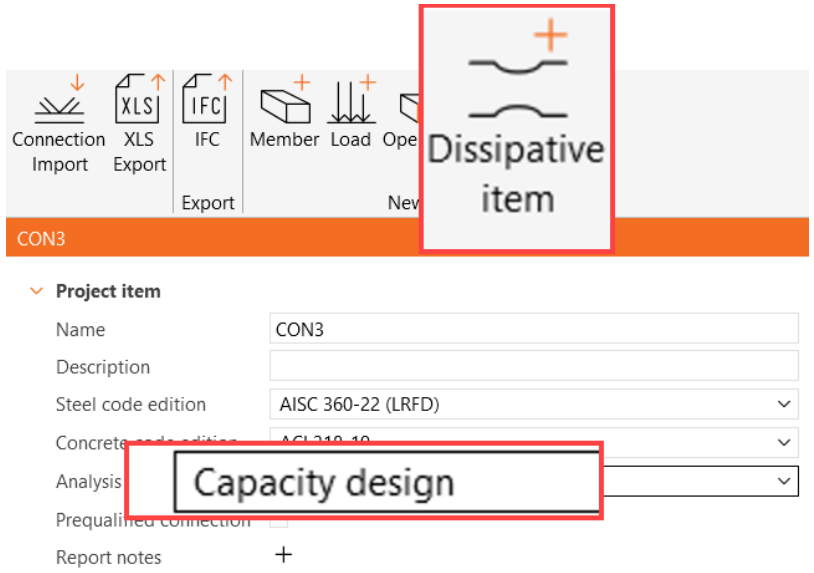
Stiffness analysis

Gusset design: Linear and elliptical hinge zone, hinge plate

Advanced operations to model special gusset geometry, parametric design

Design the brace end condition to maintain its integrity as the brace buckles

Buckling analysis



# AISC 341

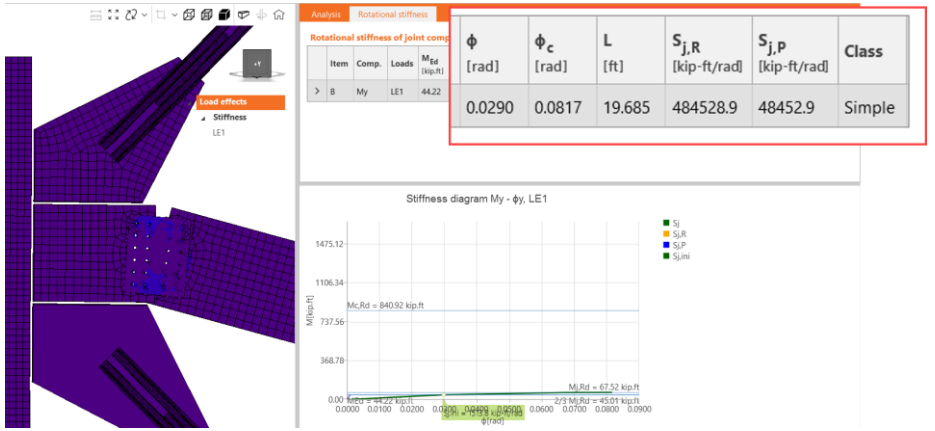
# IDEA StatiCa

Design connection elements for expected strength

Capacity design

Design beam to column connection to accommodate demands corresponding to large drifts

Stiffness analysis



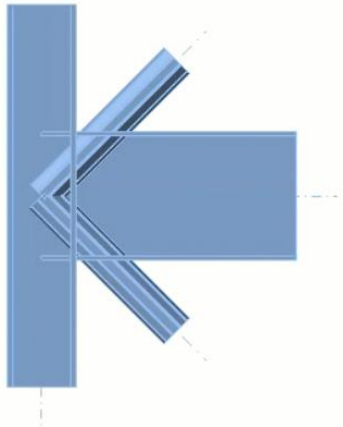

Gusset design: Linear and elliptical hinge zone, hinge plate

Advanced operations to model special gusset geometry, parametric design

Design the brace end condition to maintain its integrity as the brace buckles

Buckling analysis

| AISC 341 | IDEA StatiCa |
| --- | --- |
| Design connection elements for expected strength | Capacity design |
| Design beam to column connection to accommodate demands corresponding to large drifts | Stiffness analysis |
| Gusset design: Linear and elliptical hinge zone, hinge plate | Advanced operations to model special gusset geometry, parametric design |
| Design the brace end condition to maintain its integrity as the brace buckles | Buckling analysis |



# AISC 341

# IDEA StatiCa

Design connection elements for expected strength

Capacity design

Design beam to column connection to accommodate demands corresponding to large drifts

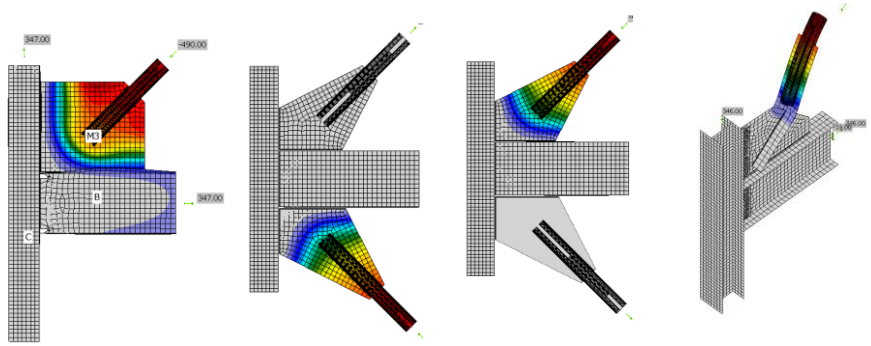
Stiffness analysis

Gusset design: Linear and elliptical hinge zone, hinge plate

Advanced operations to model special gusset geometry, parametric design

Design the brace end condition to maintain its integrity as the brace buckles

Buckling analysis



# CHECKLIST

1. Capacity design needs a yield member selection
2. Locate the maximum probable moment at the hinge location on the beam
3. Balance the moment in the column using a shear force and force position to calculate the moment at the column center line
4. Modify resistance factors if needed
5. Braced frames have a different design philosophy, review last year webinar
6. IDEA StatiCa should be used in tandem with the AISC Seismic provisions, and AISC 358.

# STUDY TOOLKIT FOR SEISMIC CONNECTION DESIGN

A repository of content to learn about seismic design in IDEA StatiCa

Bookmark it!

## Learn

- Capacity design theoretical background
- AISC 358 Prequalified connections blog post
- Detailing checks for prequalified connections
- Tutorial: Reduced beam section connection analysis

## Watch

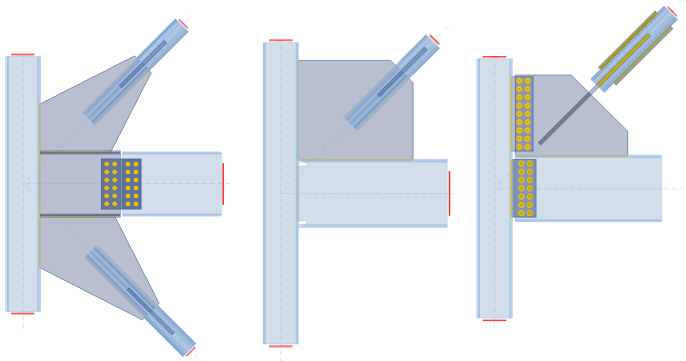
- Webinar: Special concentric braced frame connections
- Webinar: IDEA StatiCa verification for prequalified moment connections by The Ohio State University
- Video: Prequalified connections live demo

## Validate

- Prequalified Steel Connections (AISC): A verification study by The Ohio State University
- New Verification Studies: Seismic Braced Frame Connections with University of Illinois Chicago

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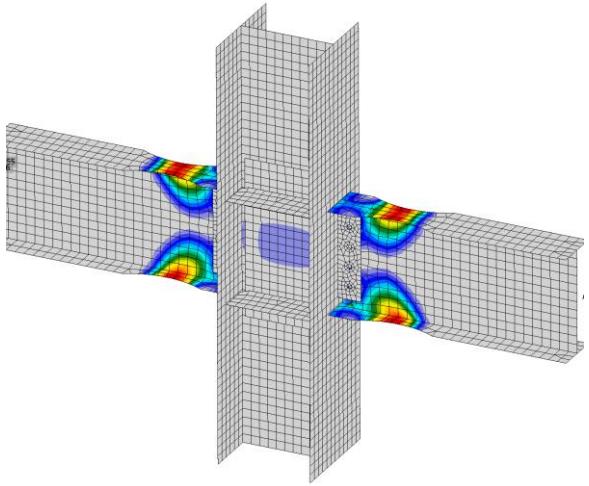
# Q&A – NEXT STEPS



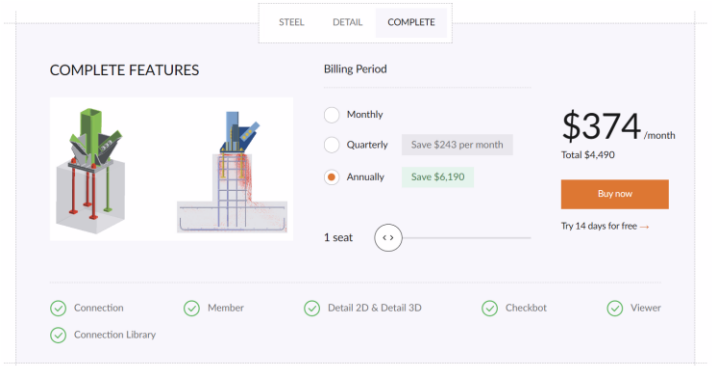
More information or questions?

Schedule a **demo**

office@ideastatica.com

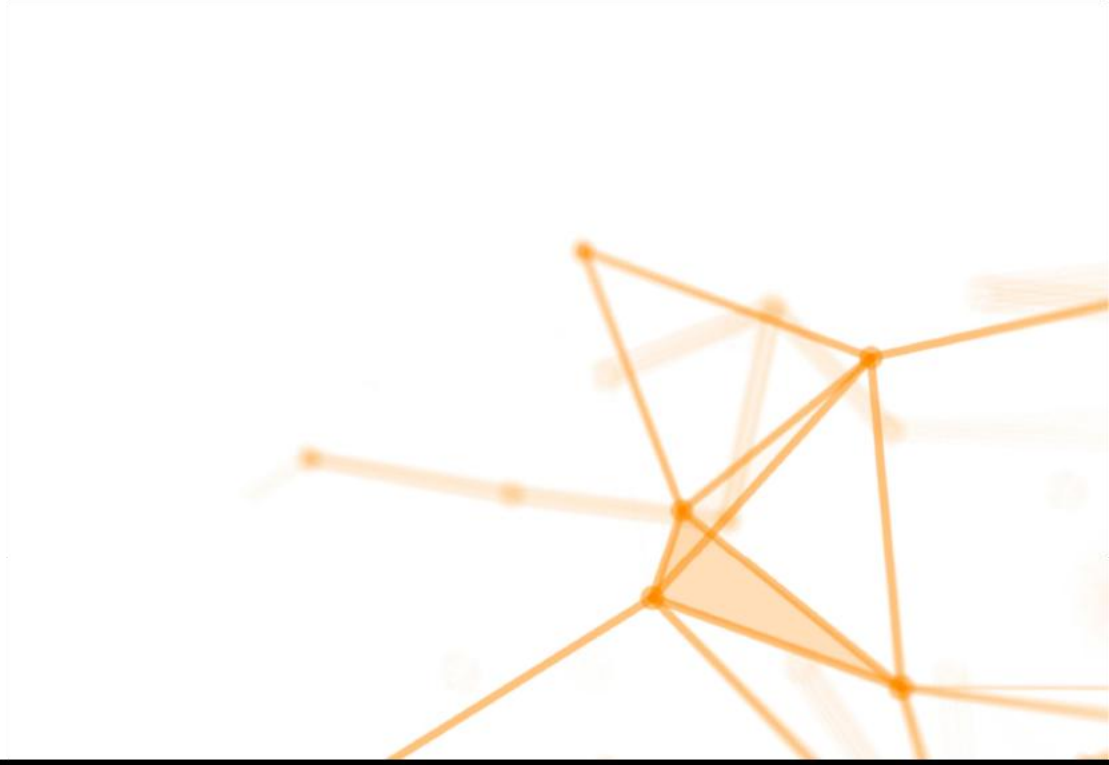


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# APPENDIX SLIDES



# DETAILING CHECKS INCLUDED IN IDEA STATICA

AISC 358, AISC 341

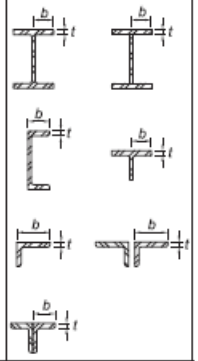
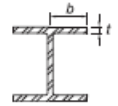
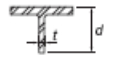
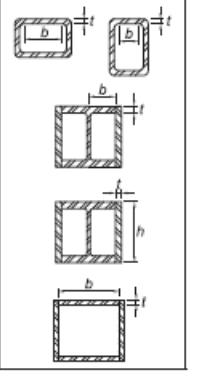


# General checks

| General check | Code | Reference |
|---|---|---|
| Limiting Width-to-thickness ratios | AISC 341-16 | Chapter D1.1.b Table D.1.1 |
| Type of member's section | AISC 358-16 | Chapter 2.3 |
| Fastening assemblies | AISC 358-16 | Chapter 4.1 |
| Protected zone | AISC 341-16 | Chapter D1.3 |
| Geometry of Continuity plates | AISC 341-16 | Chapter E2.6f.2, E3.6f.2 |
| Geometry Doubler plates | AISC 341-16 | E3.6e |



**TABLE D1.1**
**Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members**

| | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratio: $\lambda_{hd}$ Highly Ductile Members | Limiting Width-to-Thickness Ratio: $\lambda_{md}$ Moderately Ductile Members | Example |
|---|---|---|---|---|---|
| Unstiffened Elements | Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double-angle members with separators; outstanding legs of pairs of angles in continuous contact | $b/t$ | $0.32\sqrt{\frac{E}{R_yF_y}}$ | $0.40\sqrt{\frac{E}{R_yF_y}}$ |  |
| | Flanges of H-pile sections per Section D4 | $b/t$ | not applicable | $0.48\sqrt{\frac{E}{R_yF_y}}$ |  |
| | Stems of tees | $d/t$ | $0.32\sqrt{\frac{E}{R_yF_y}}$ [a] | $0.40\sqrt{\frac{E}{R_yF_y}}$ |  |
| Stiffened Elements | Walls of rectangular HSS used as diagonal braces | $b/t$ | $0.65\sqrt{\frac{E}{R_yF_y}}$ | $0.76\sqrt{\frac{E}{R_yF_y}}$ |  |
| | Flanges of boxed I-shaped sections | $b/t$ | | | |
| | Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces | $h/t$ | | | |
| | Flanges of built-up box shapes used as link beams | $b/t$ | | | |

*Seismic Provisions for Structural Steel Buildings*, July 12, 2016
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

# Reduced beam section (RBS)

| Code checks | Code | Chapter |
|---|---|---|
| Beam limitations | AISC 358-16 | Chapter 5.3.1. |
| Column limitations | AISC 358-16 | Chapter 5.3.2. |
| Beam-flange to column weld | AISC 358-16 | 5.5 |
| Weld access hole | AISC 360-16 | J1.6 |
| Beam-web to column weld limitations | AISC 358-16 | 5.6 |
| Geometry of the RBS | AISC 358-16 | 5.8 Step 1 |
| The shear strength according to the Chapter 5.6 (1) and weld of fin plate according to the Chapter 5.6 (2) are stated as out of scope the check in Connection. | | |

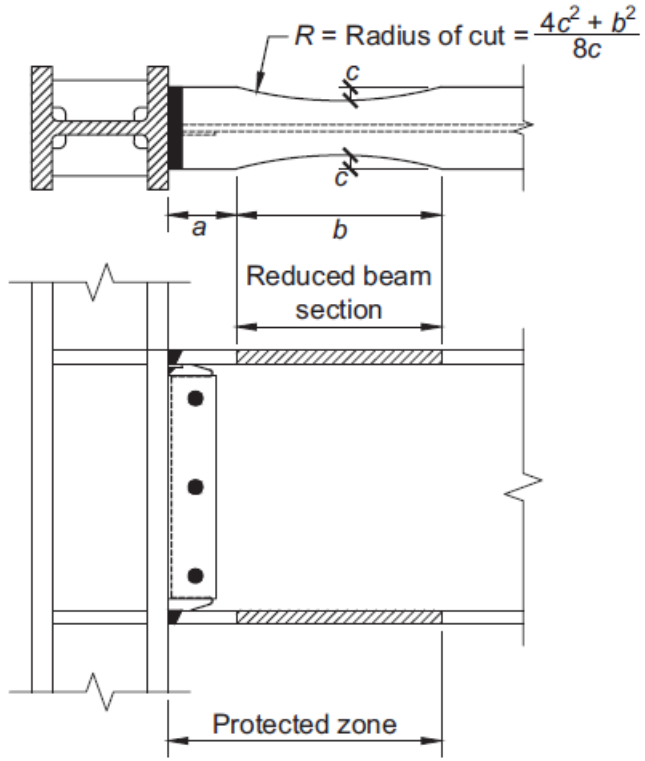
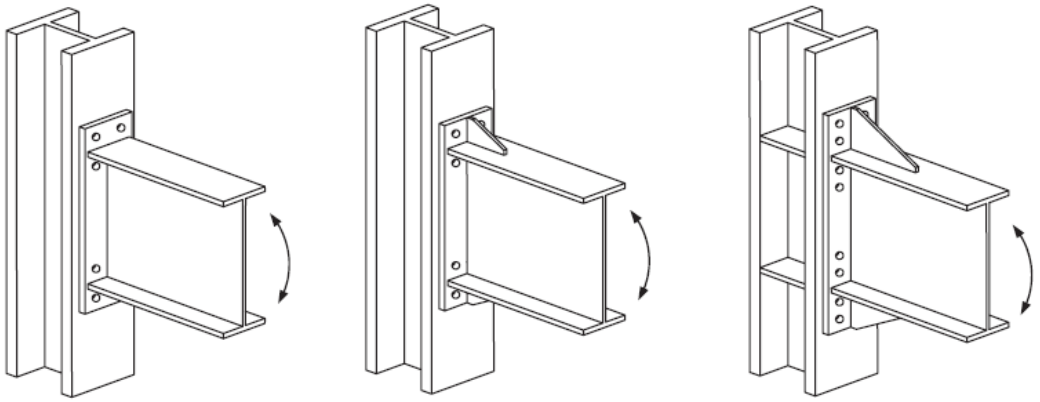



*Fig. 5.1. Reduced beam section connection.*

# Extended plate

| Code checks | Code | Chapter |
|---|---|---|
| Beam limitations | AISC 358-16 | Chapter 6.3.1. |
| Column limitations | AISC 358-16 | Chapter 6.3.2. |
| Connection detailing limitations (bolt, end plate and stiffener geometry) | AISC 358-16 | 6.7, 6.1 |

Finger shims are not supported in the operation and are stated as out of scope.
Also check of the welds for the full strength of the beam web in tension according to the Chapter 6.7.6 (3) is stated as out of scope.



(a) (b) (c)

*Fig. 6.1. Extended end-plate configurations: (a) four-bolt unstiffened, 4E; (b) four-bolt stiffened, 4ES; (c) eight-bolt stiffened, 8ES.*

# Bolted flange plates

| Code checks | Code | Chapter |
|---|---|---|
| Beam limitations | AISC 358-16 | Chapter 7.3.1. |
| Column limitations | AISC 358-16 | Chapter 7.3.2. |
| Connection detailing of plates material, welds and bolt grade and diameter | AISC 358-16 | 7.5 |

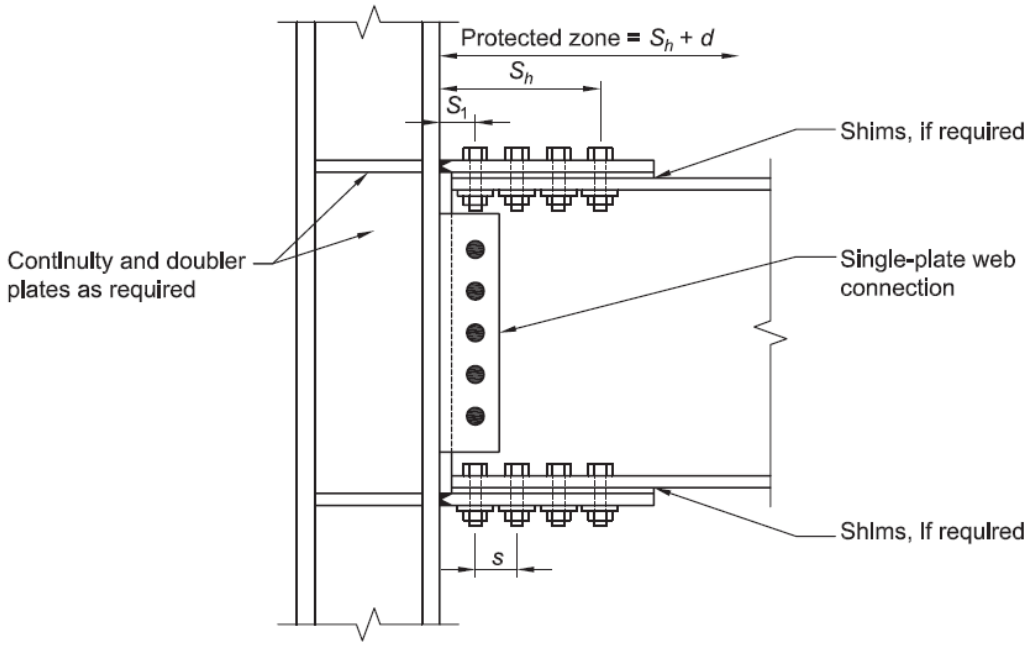



*Fig. 7.1. Bolted flange plate moment connection.*

# Welded unreinforced flange welded web

| Code checks | Code | Chapter |
|---|---|---|
| Beam limitations | AISC 358-16 | Chapter 8.3.1. |
| Column limitations | AISC 358-16 | Chapter 8.3.2. |
| Beam-flange to column weld | AISC 358-16 | 8.5 |
| Beam-web to column weld | AISC 358-16 | 8.6 |
| The weld of single plate to column according to the Chapter 5.6 (2) is stated as out of scope the check in Connection. | | |

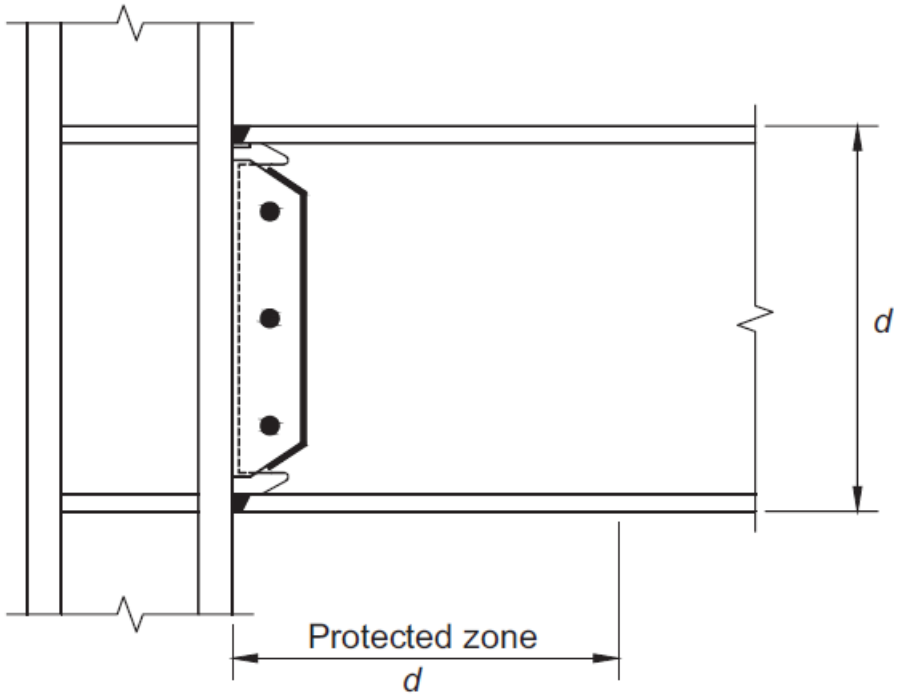



*Fig. 8.1. WUF-W moment connection.*

# Double tee

| Code checks | Code | Chapter |
|---|---|---|
| Beam limitations | AISC 358-16 | Chapter 13.3.1. |
| Column limitations | AISC 358-16 | Chapter 13.3.2. |
| Connection detailing: continuity plates, welds and bolts grade and diameter | AISC 358-16 | 13.5 |

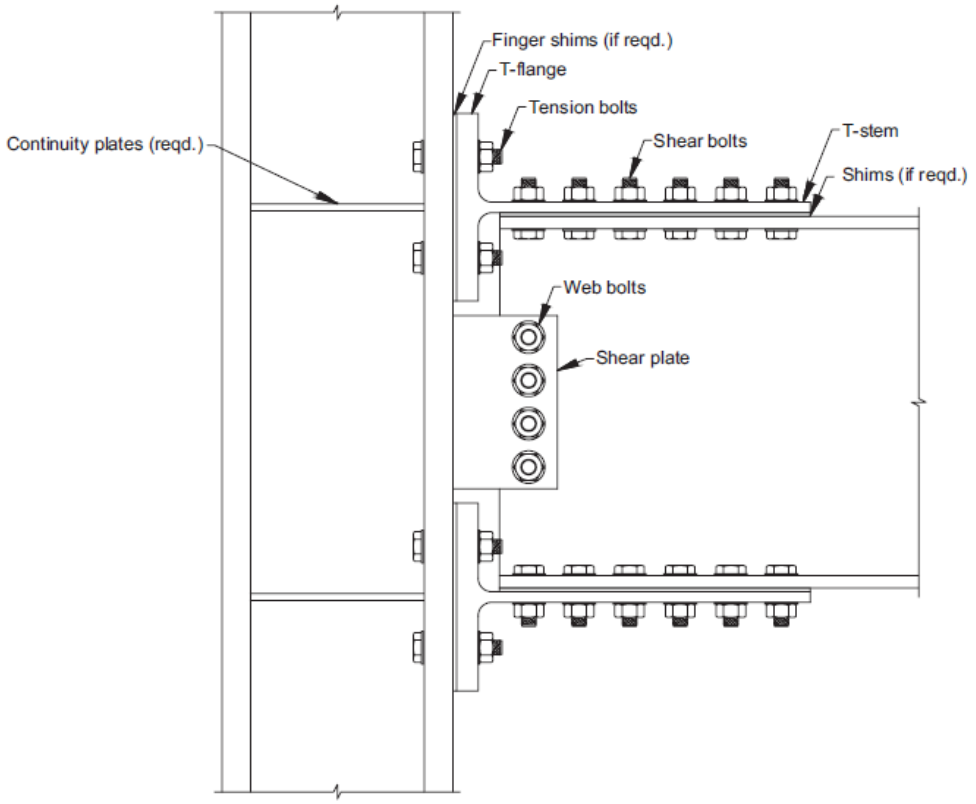
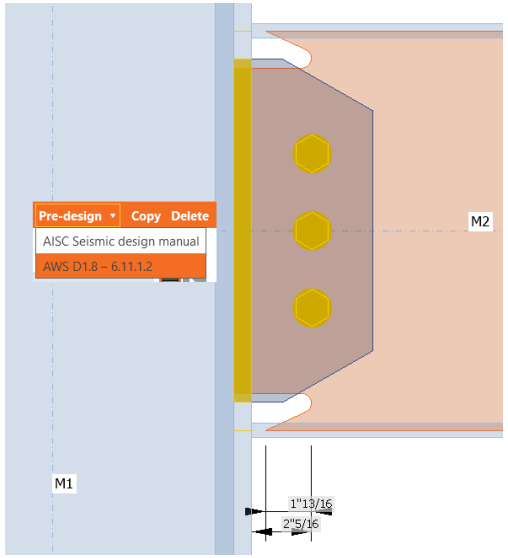
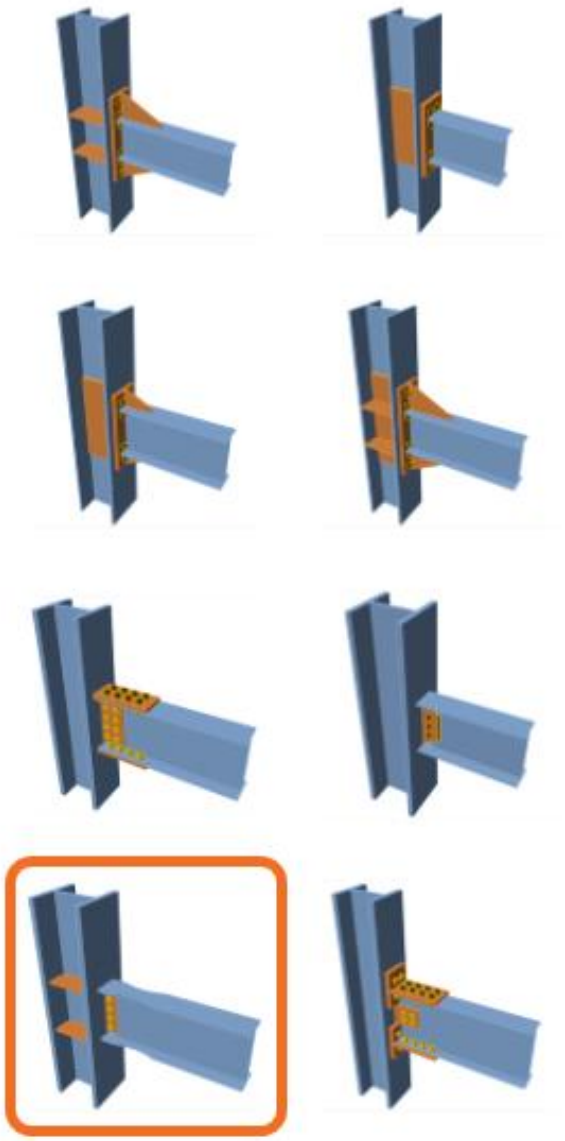
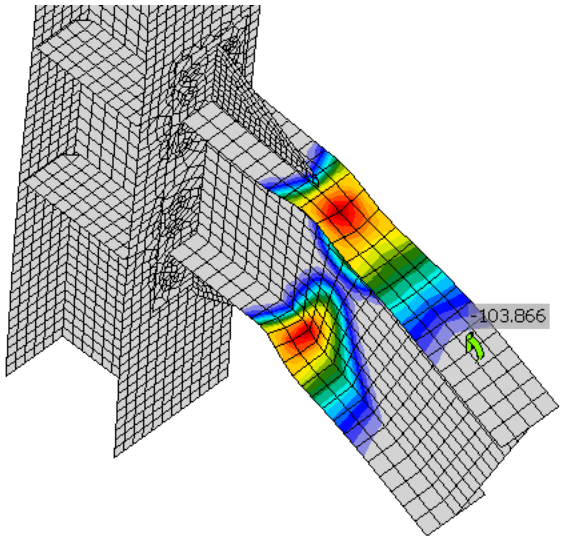
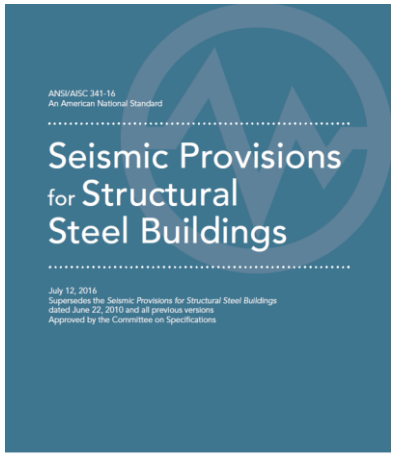



*Fig. 13.1. Typical double-tee connection.*

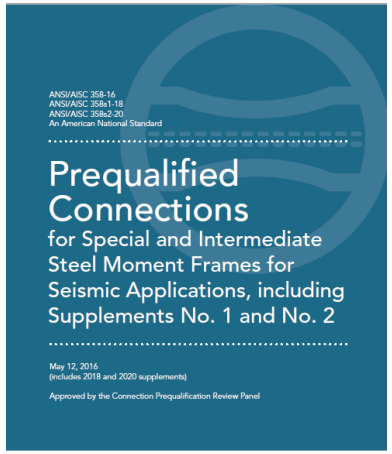


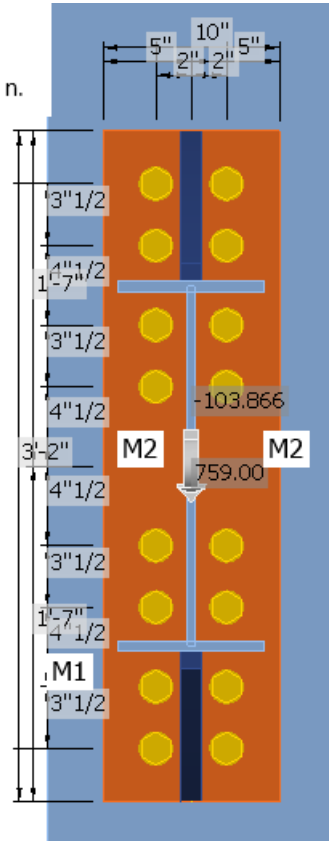



Smarter. Stronger. Steel.




Smarter. Stronger. Steel.




**Prequalification limits**

End plate EP1 bolt grid horizontal gage 4"

shall be ≥ 5" in and ≤ 6" in.

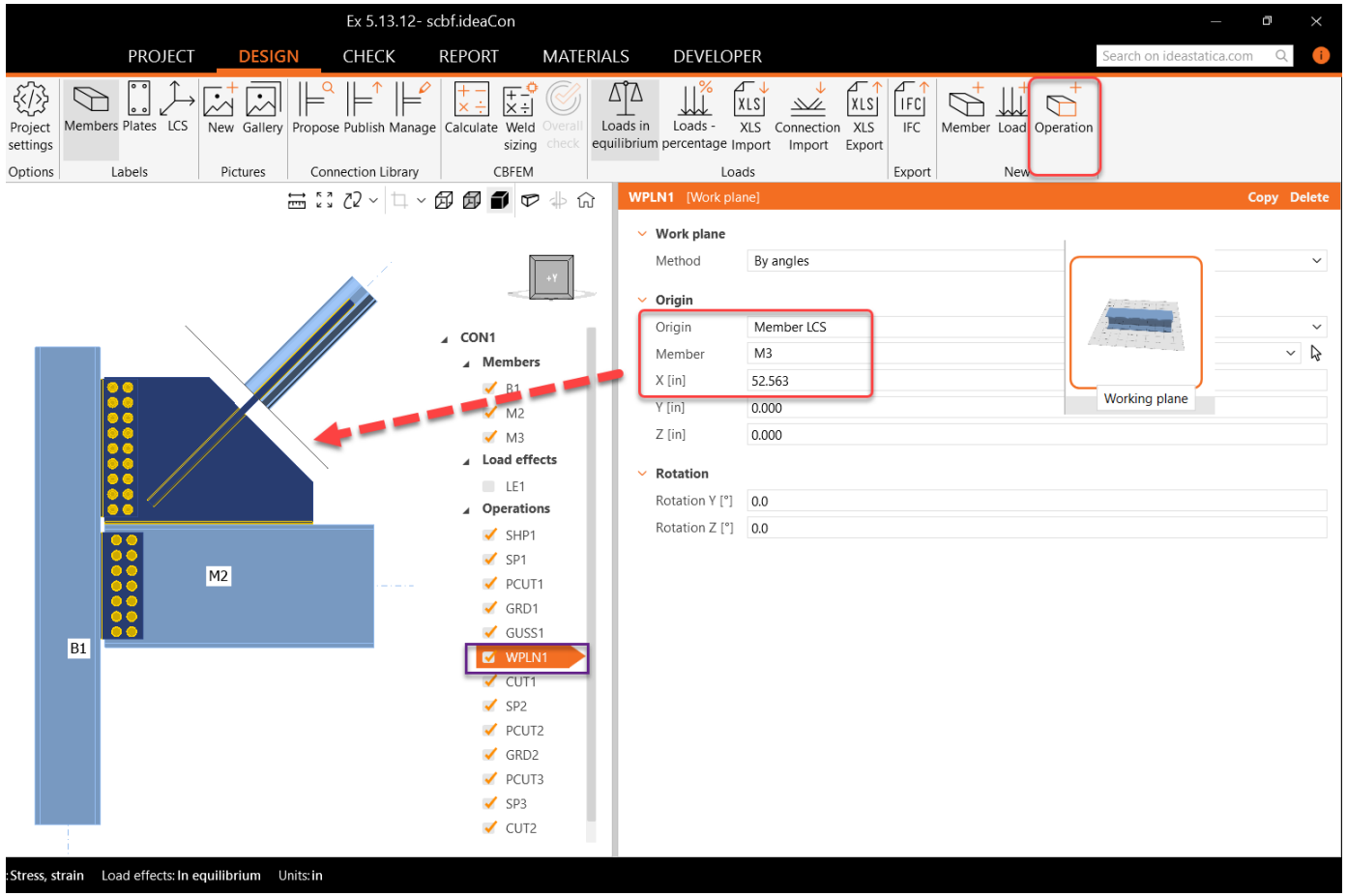
# APPENDIX

Tips for modeling SCBF Connections in IDEA StatiCa



# WORKING PLANES FROM MEMBER LOCAL AXIS

The working plane operation help us to pull off the member from the node

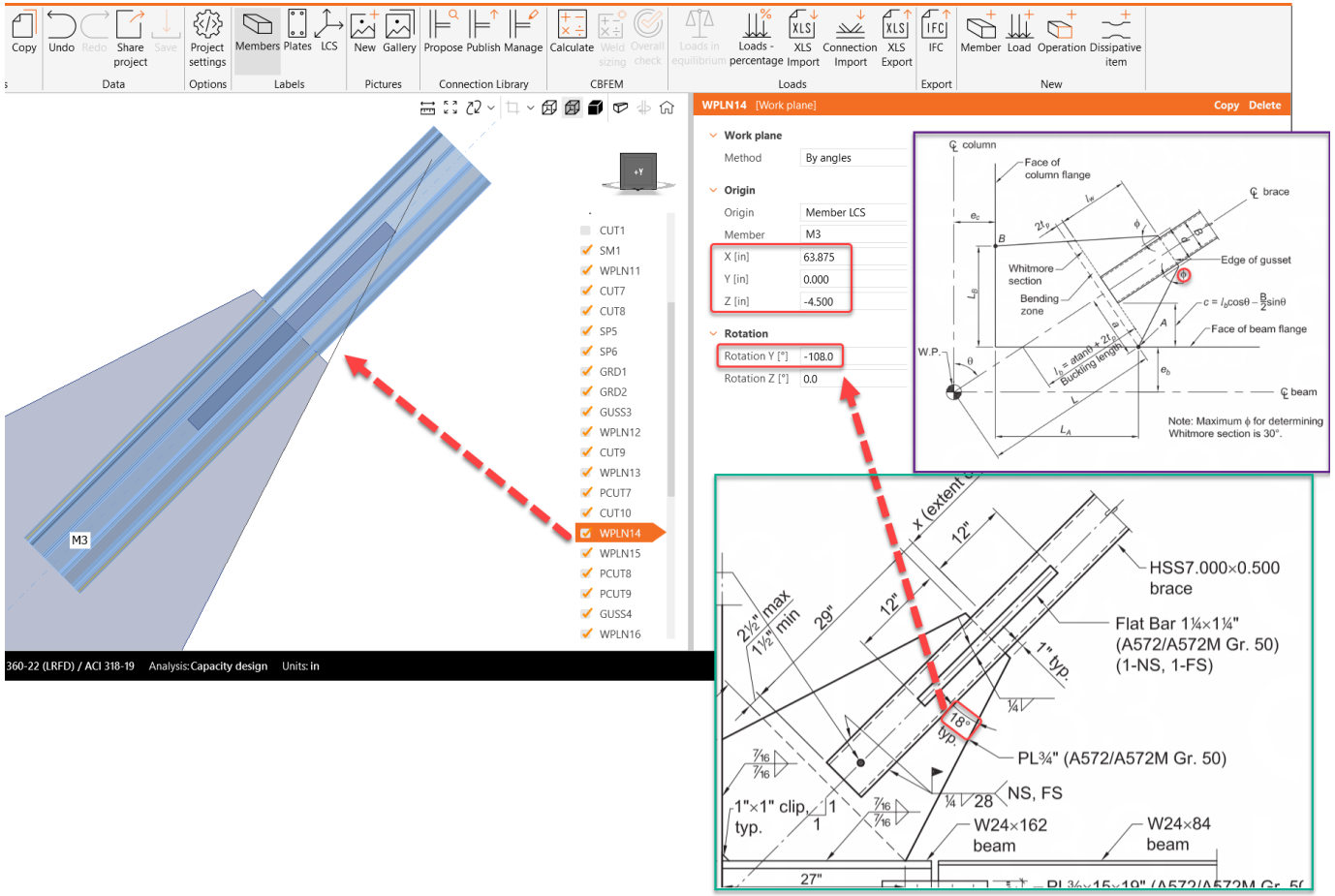


# GUSSET PLATE TAPERED DESIGN

Working planes from the member local axis

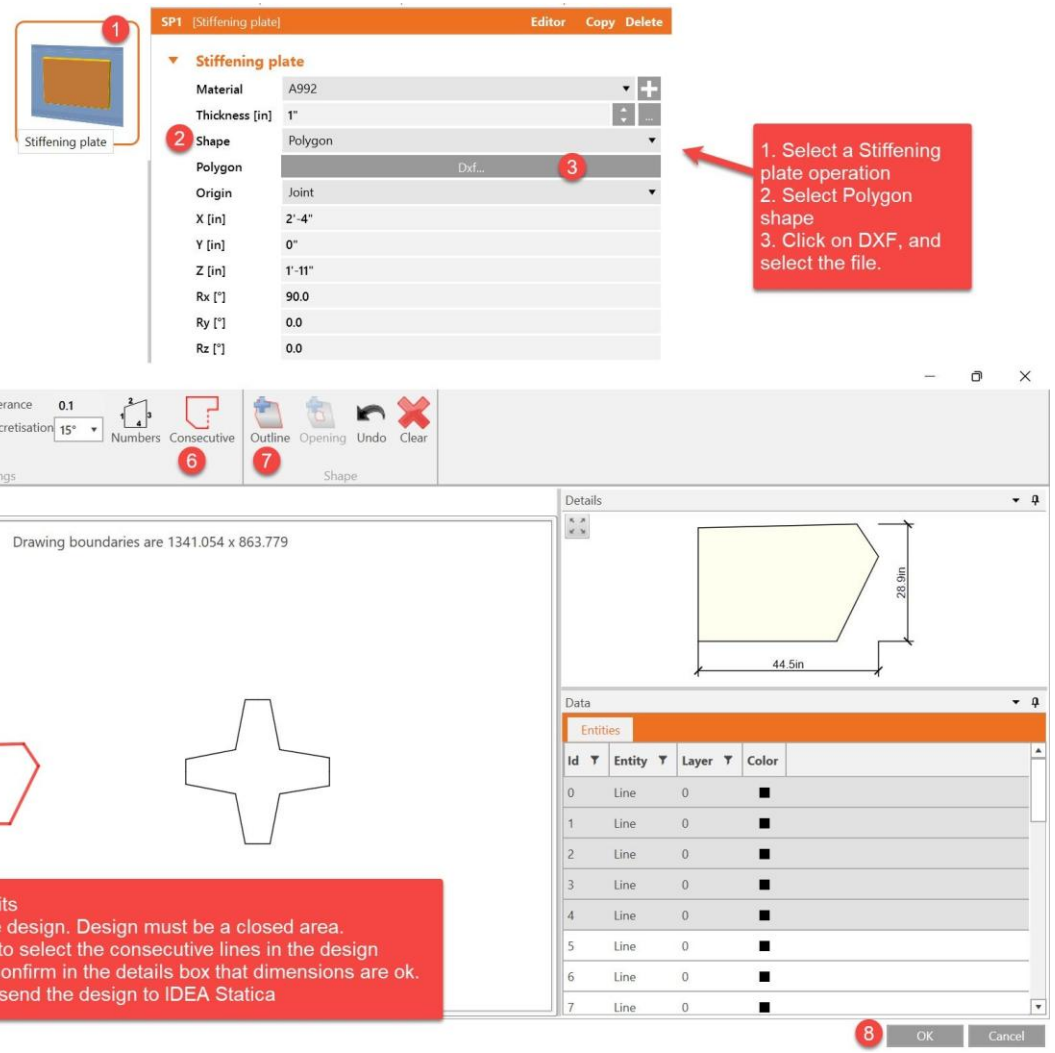
Use angle rotations

Later use a plate cut to form the tapered gusset plate



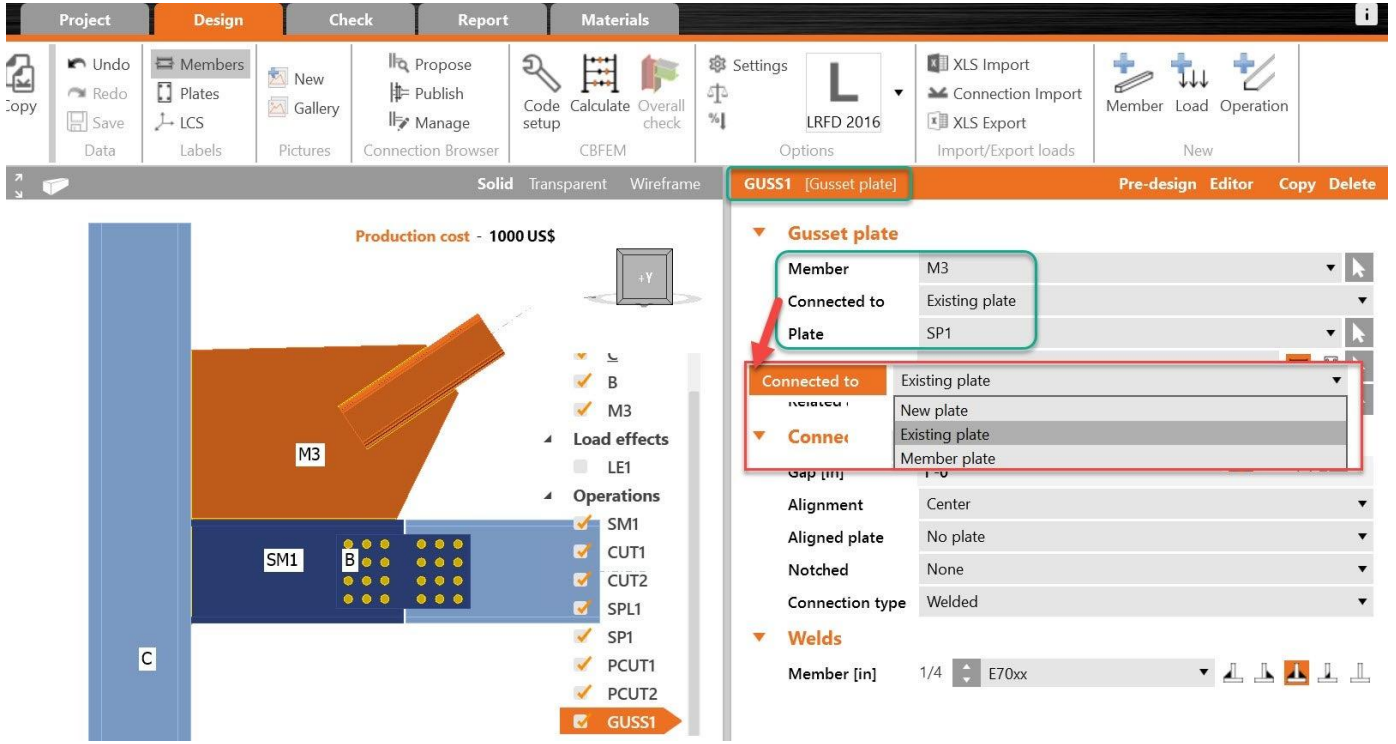
# PLATE SHAPE IMPORT FROM A CAD FILE

Import a DXF drawing with a specific plate design and use it for the gusset plate



# GUSSET PLATE OPERATION: EXISTING PLATE

When using a gusset plate operation, the operation itself can create a new plate, but if there is the case that the plate is already in the model, the option of the **existing plate** in the model.



# STIFFNESS ANALYSIS IN IDEA STATICA
## STEP BY STEP

Tutorial

1. Perform Stress/strain analysis first, to ensure correct modeling

2. Copy the connection and change the analysis type to Stiffness analysis

3. Select the analyzed member, only one member in the joint can be analyzed

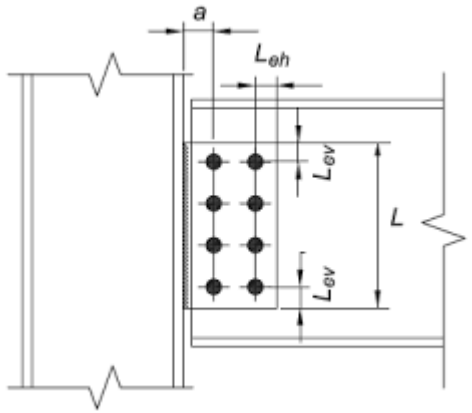
4. Input moment force/axial force, depending what value of stiffness is needed. It is recommended to input only one force in the analyzed member

5. Calculate and review results

IDEA StatiCa®

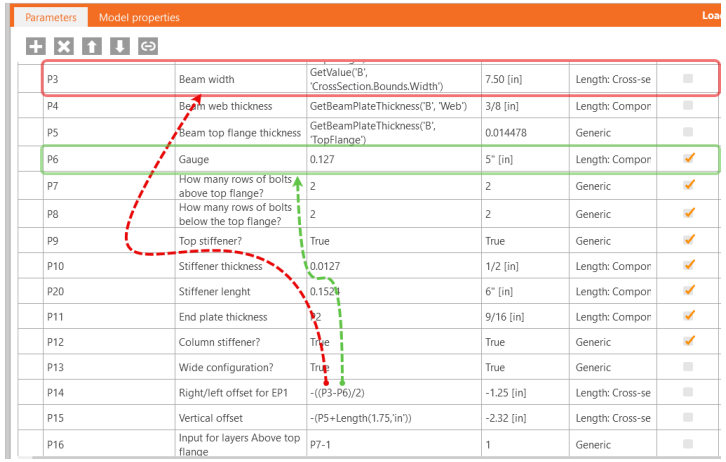
# PARAMETRIC TEMPLATES

Tutorial: Parametric design in IDEA StatiCa Connection - Flush moment end plate connections

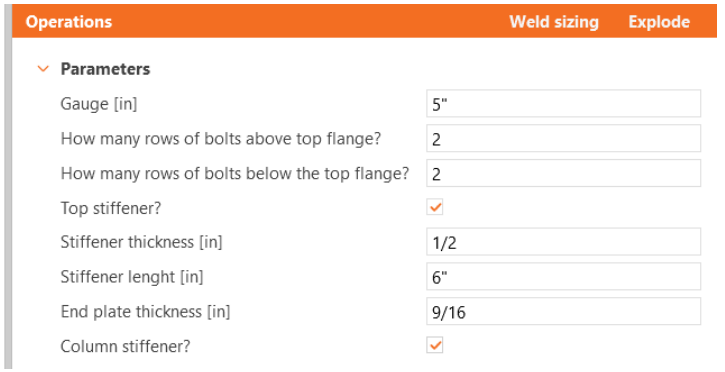



Custom company connections

Common company details




Smart templates

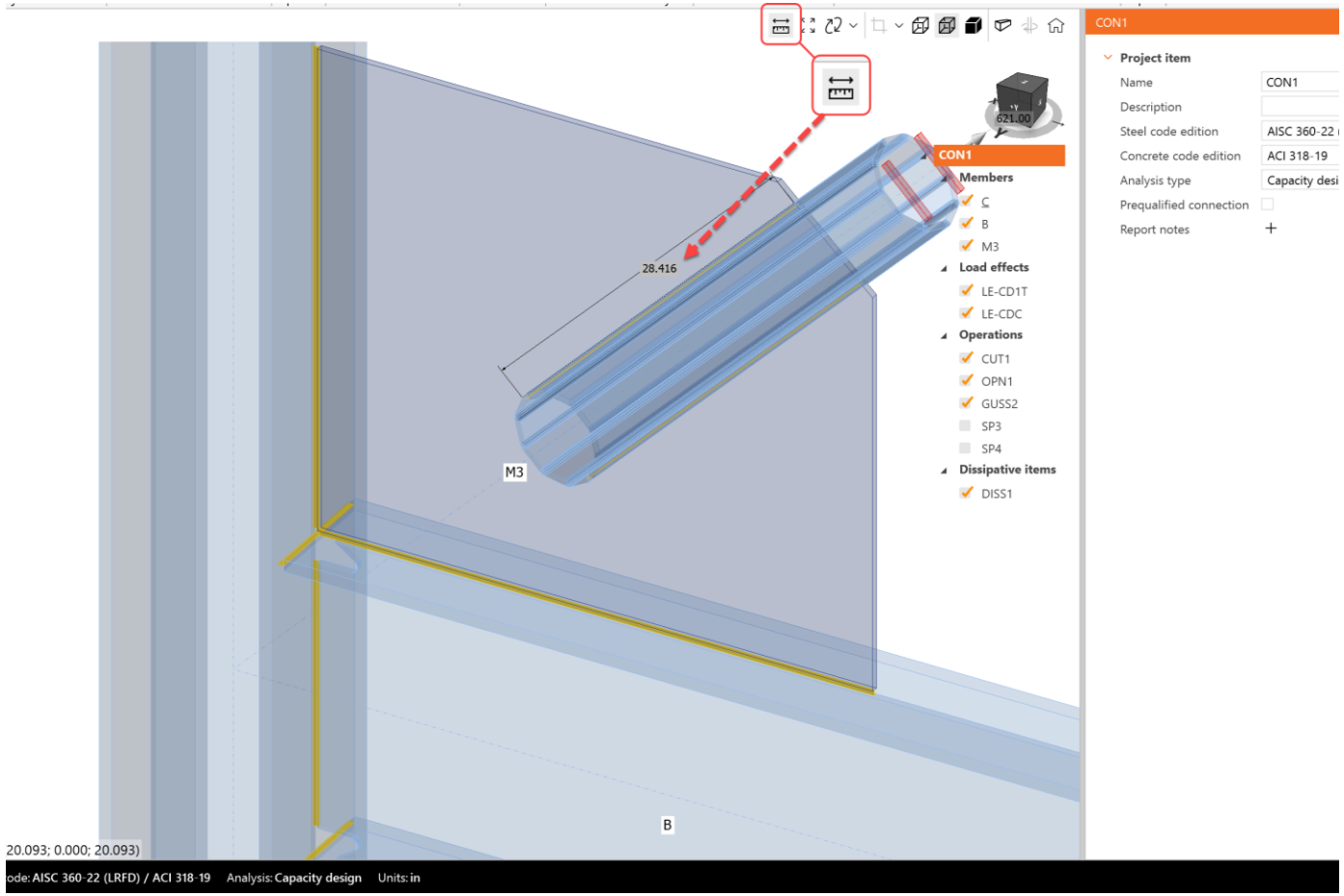



Avoid learning curve for simple connections

IDEA StatiCa®

# MEASURE TOOL

Use the measure tool to get distances between points, edges and surfaces

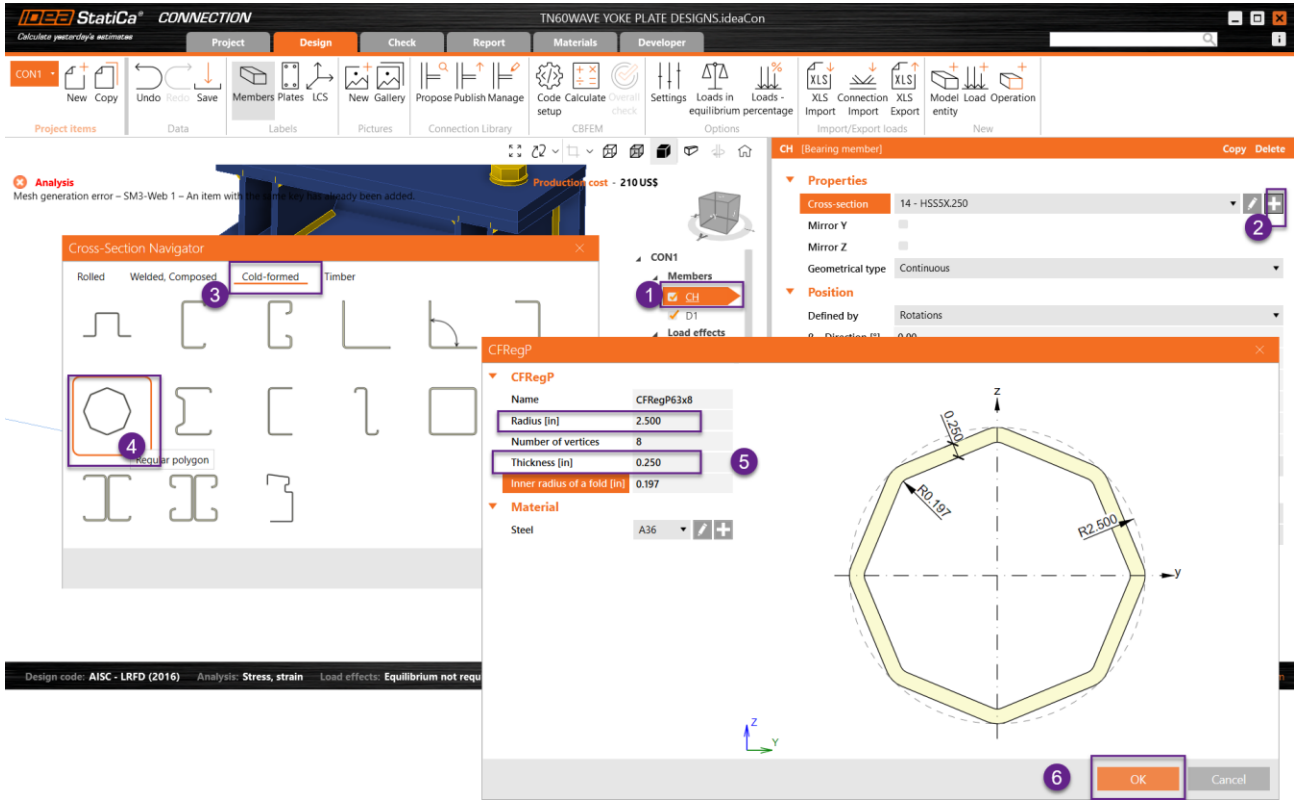


# USE OF POLYGON SHAPE TO REINFORCE THE MEMBER

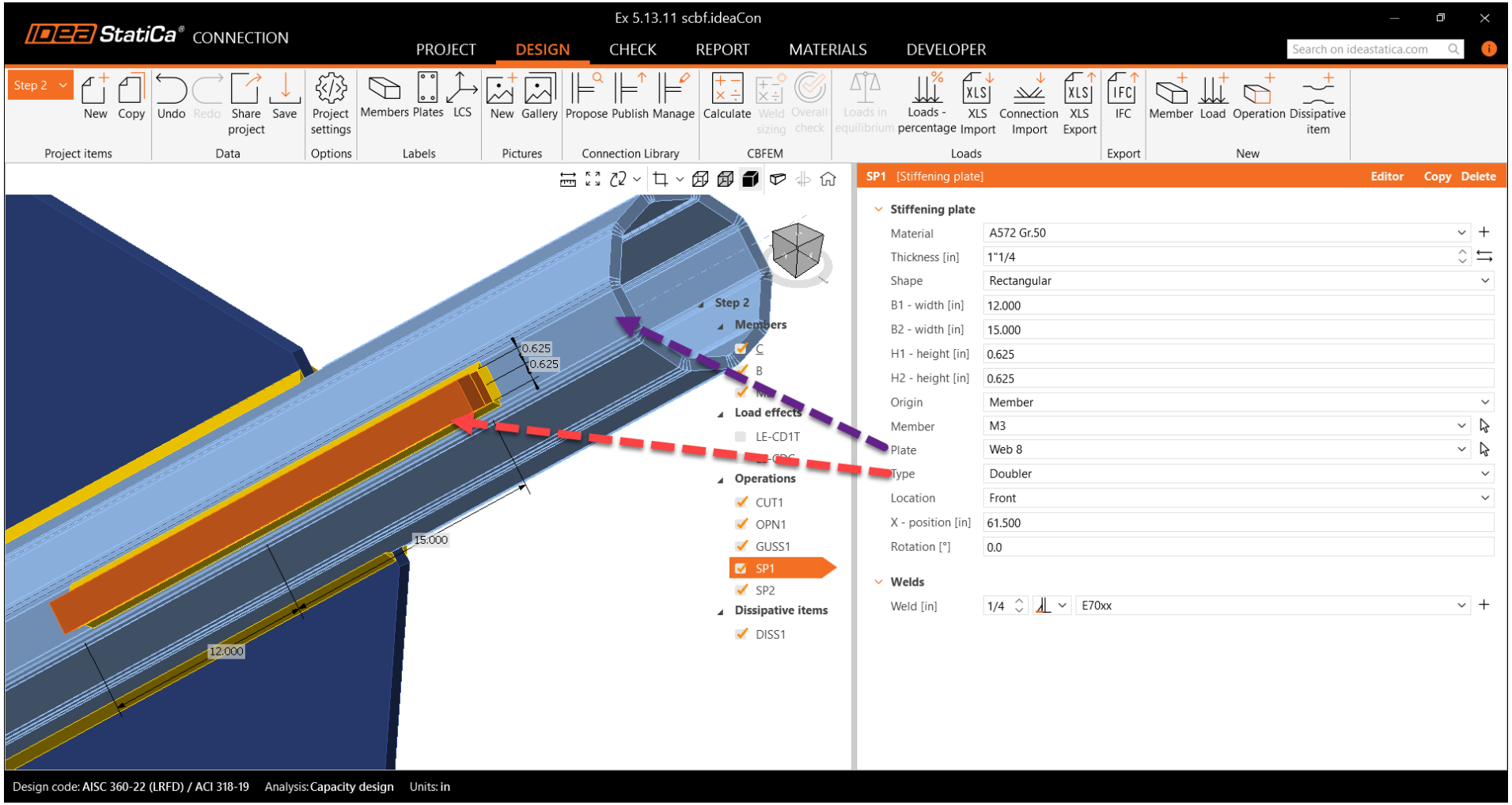
When using HSS sections, the members are break out in vertical thin plate strips.

However, you cannot place HSS reinforcement in those stips. The workaround is to use a hollow section polygon under the cold-formed database. With that type of section, the plates are workable.

Note: Make sure you input the correct design thickness

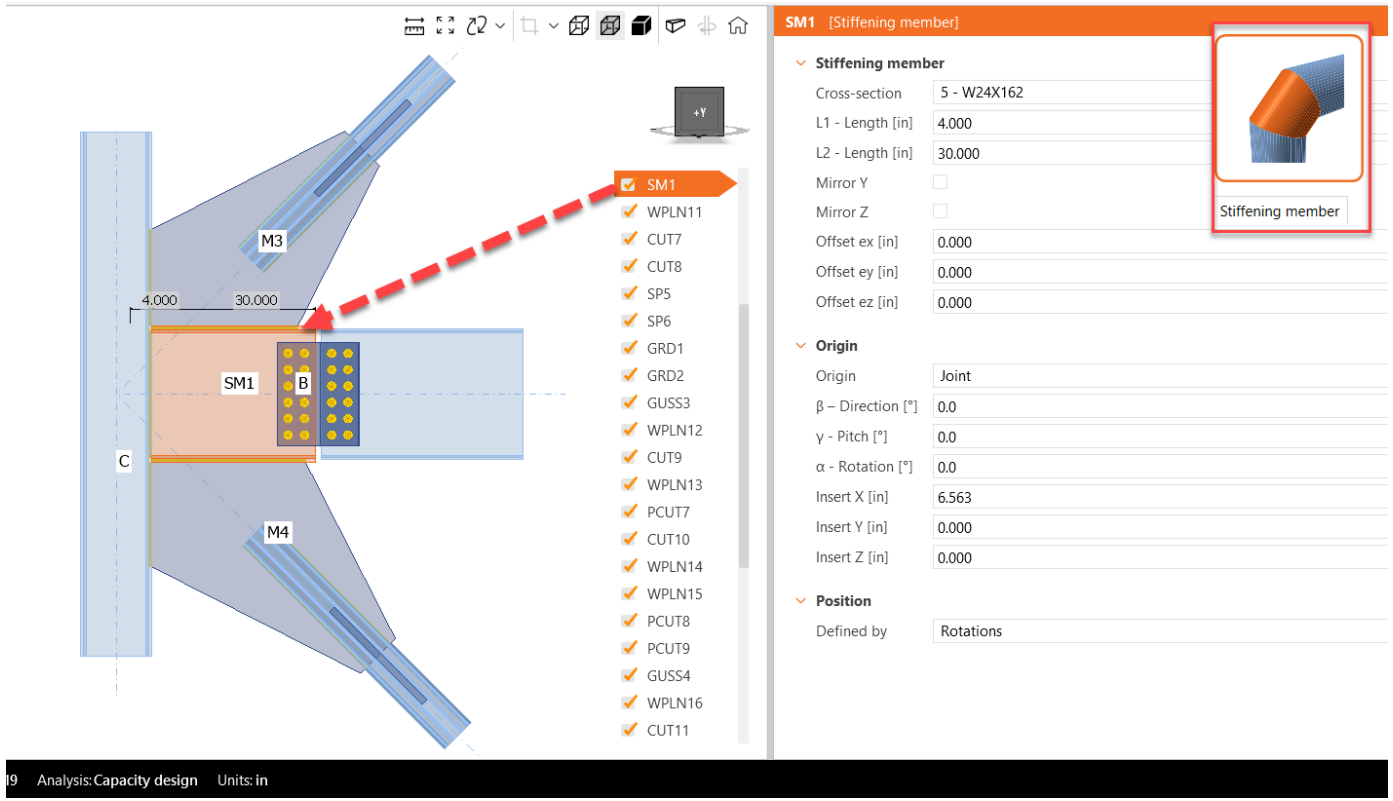


# REINFORCING PLATE: STIFFENING PLATE AS A DOUBLER

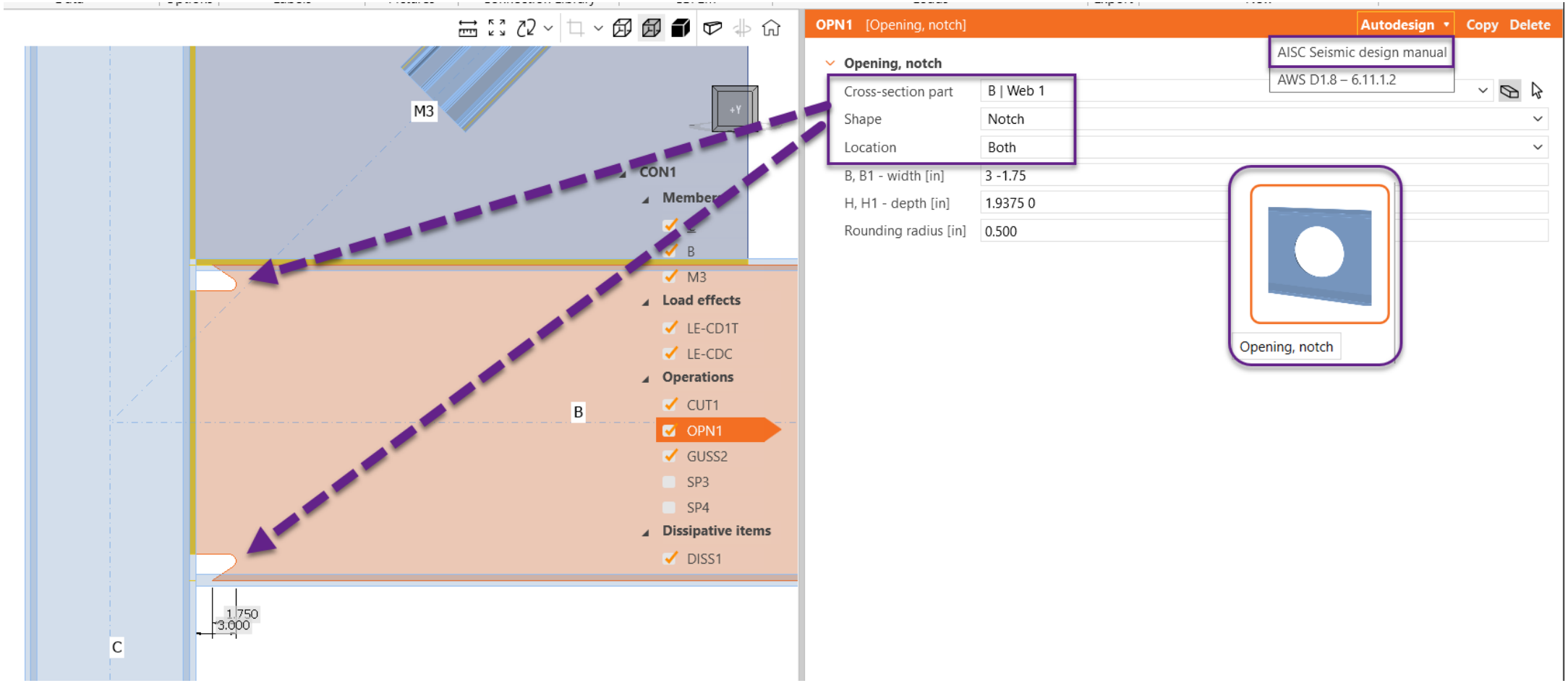


# STIFFENING MEMBER FOR SHEAR CONNECTION OUT OF THE JOINT

A stiffening member is an operation that helps to add a member in the model to be part of the connection. One of the keys to using a stiffening member is that you can not directly apply load to it. This operation helps in the next examples:

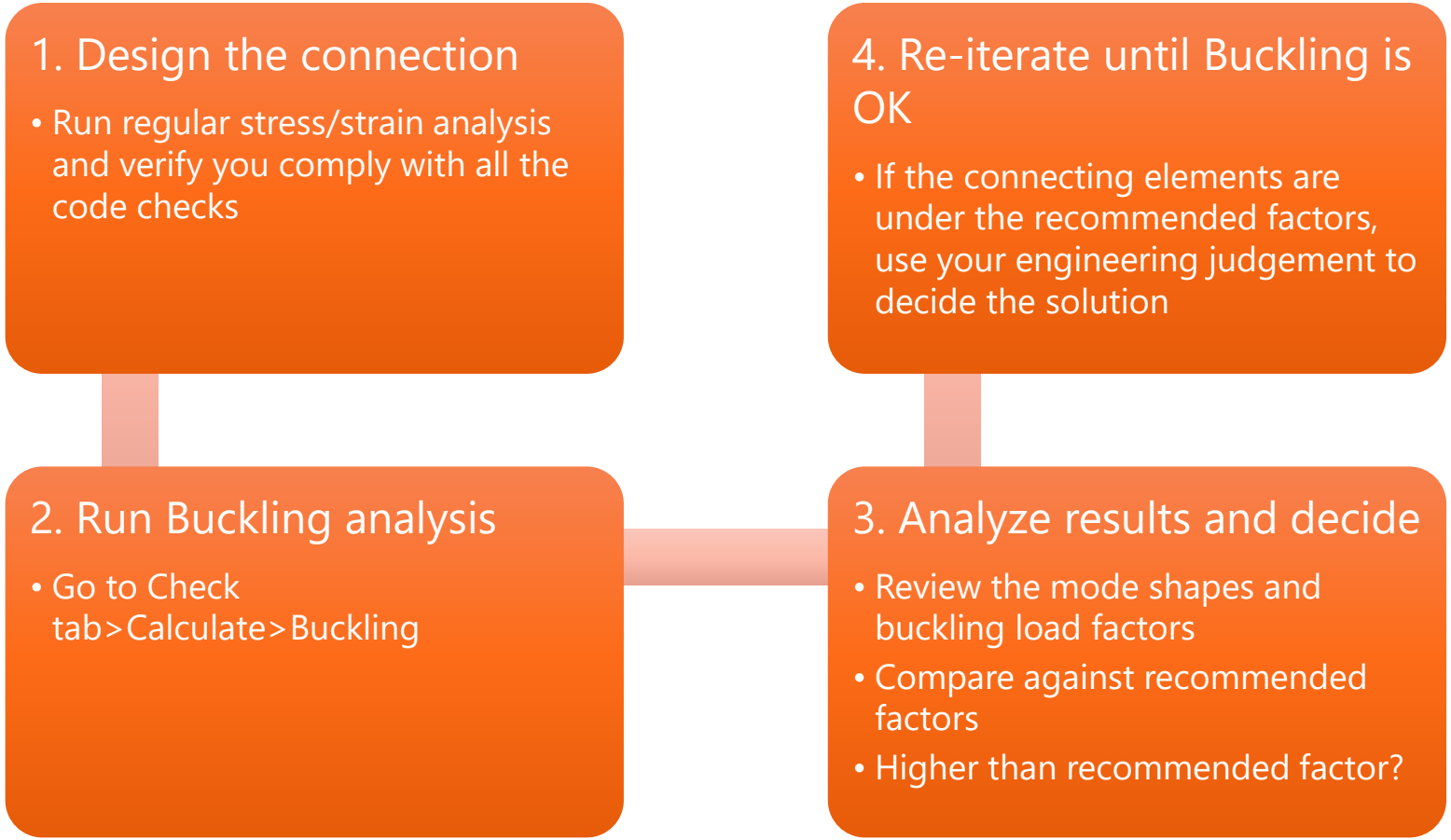


# BEAM NOTCH AUTO DESIGN: AISC



# BUCKLING ANALYSIS IN IDEA STATICA

https://www.ideastatica.com/webinars/linear-buckling-analysis-for-steel-connection-design




IDEA StatiCa®

# BUCKLING ANALYSIS SUMMARY

IDEA StatiCa connection uses linear buckling analysis to provide a buckling factor only and recommended limit factors are provided

The recommended limit factors ensure the non-slender design of connection plates

Buckling analysis in IDEA StatiCa is a tool

It doesn't provide pass/fail result

Use your engineering judgement and experience to decide the  design solution

Before running the analysis, make sure loads and the connection design is OK