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US Webinar

Linear Buckling analysis for steel connection design

June 26, 2024

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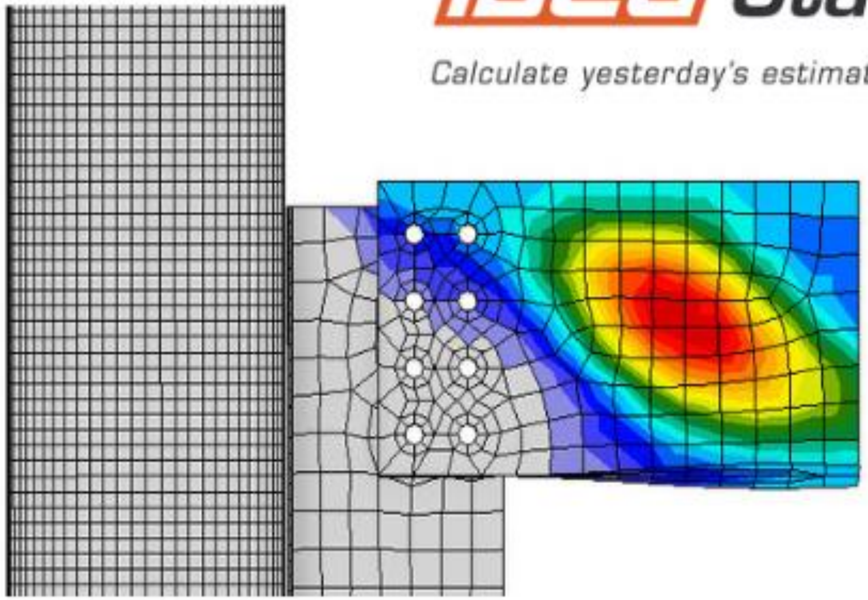

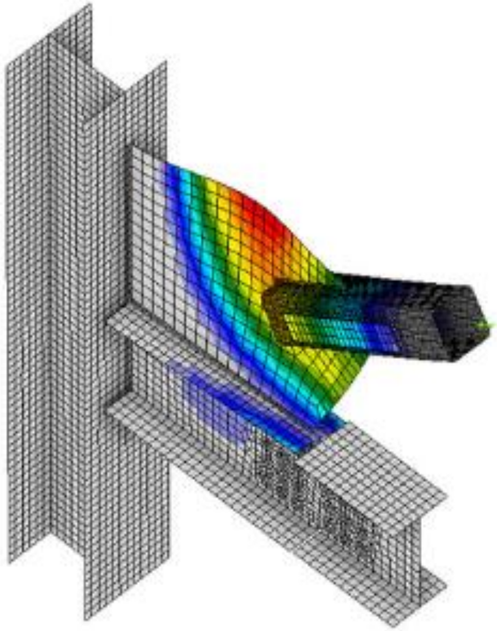

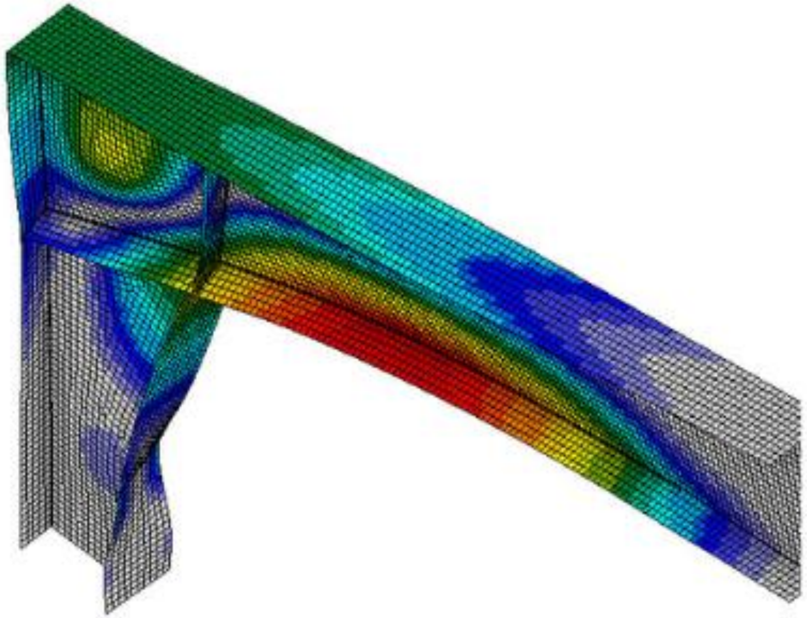

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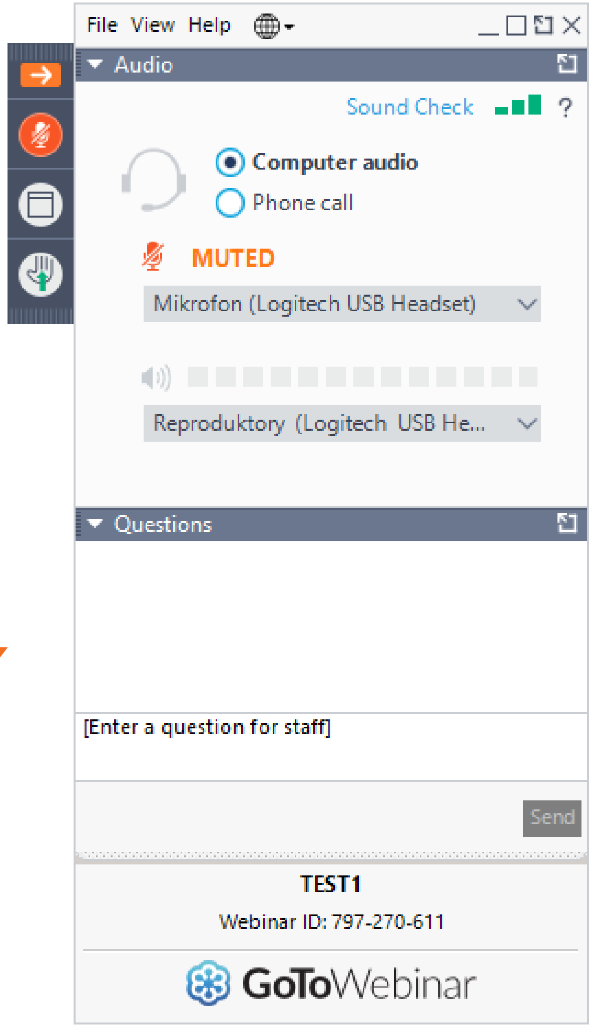
- **Grab Tab**: From the Grab Tab, you can hide the Control Panel, mute yourself (if you have been unmuted by the organizer), view the webinar in full screen and raise your hand.
- **Audio Pane**: Use the Audio pane to switch between Telephone and Mic & Speakers.
- **Questions Pane**: Ask questions for the staff.

QUESTIONS

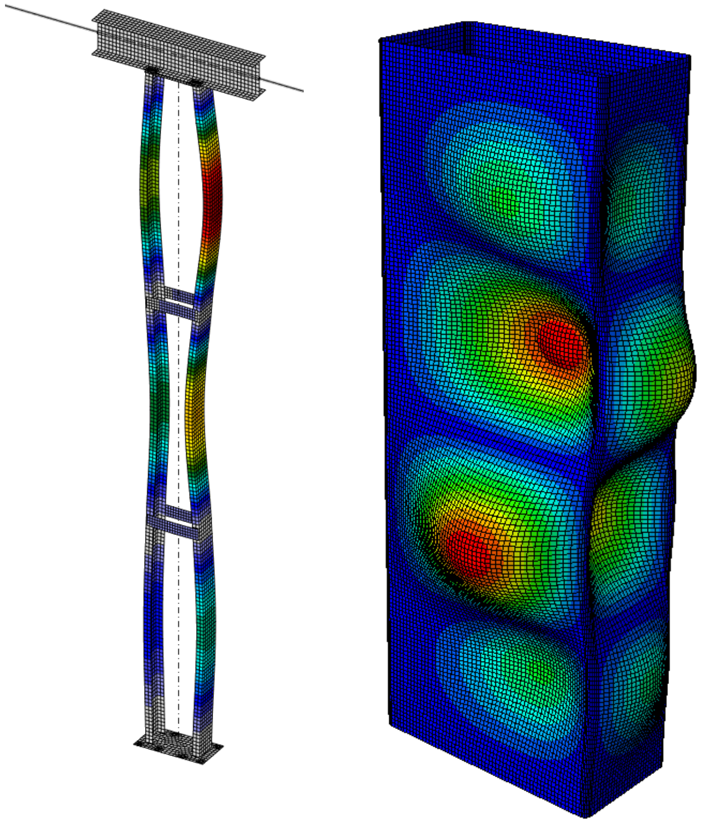
HANDOUT AVAILABLE



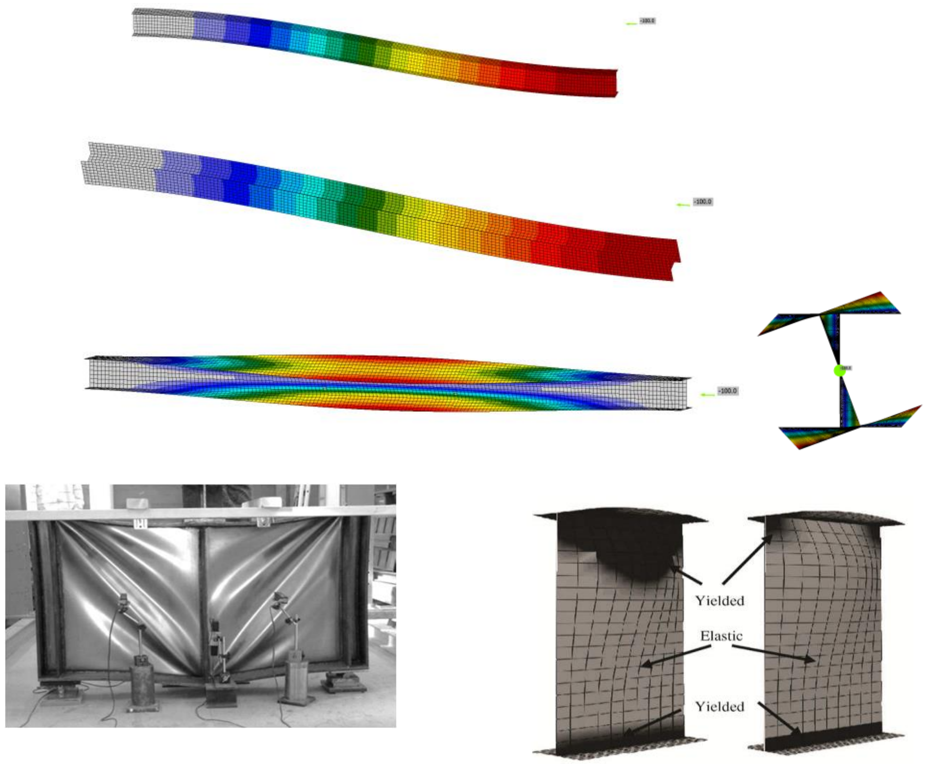
Agenda

- Linear buckling analysis
- IDEA StatiCa linear buckling analysis
- Recommended limit factors
- Examples
- Q&A

Buckling in structural engineering



Columns



Beams

Connections

Buckling limit state in connections - AISC

AISC 360 -22

- J4 - Affected elements of members and connecting elements

4. Strength of Elements in Compression

The available strength of connecting elements in compression for the limit states of yielding and buckling shall be determined as follows:

(a) When $L_c/r \leq 25$

$$P_n = F_y A_g \quad \text{(J4-6)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) When $L_c/r > 25$, the provisions of Chapter E apply,

where

L_c = effective length, in. (mm)
= KL
K = effective length factor
L = laterally unbraced length of the element, in. (mm)

User Note: The effective length factors used in computing compressive strengths of connecting elements are specific to the end restraint provided and may not necessarily be taken as unity when the direct analysis method is employed.

MANUAL AISC Chapter 9

CONNECTING ELEMENTS SUBJECT TO COMPRESSION YIELDING AND BUCKLING

When connecting elements are subject to compression, the available compressive strength, ϕP_n or P_n/Ω, which must equal or exceed the required compressive strength, P_u or P_a, respectively, is determined in accordance with AISC *Specification* Section J4.4.

AFFECTED AND CONNECTING ELEMENTS SUBJECT TO FLEXURE

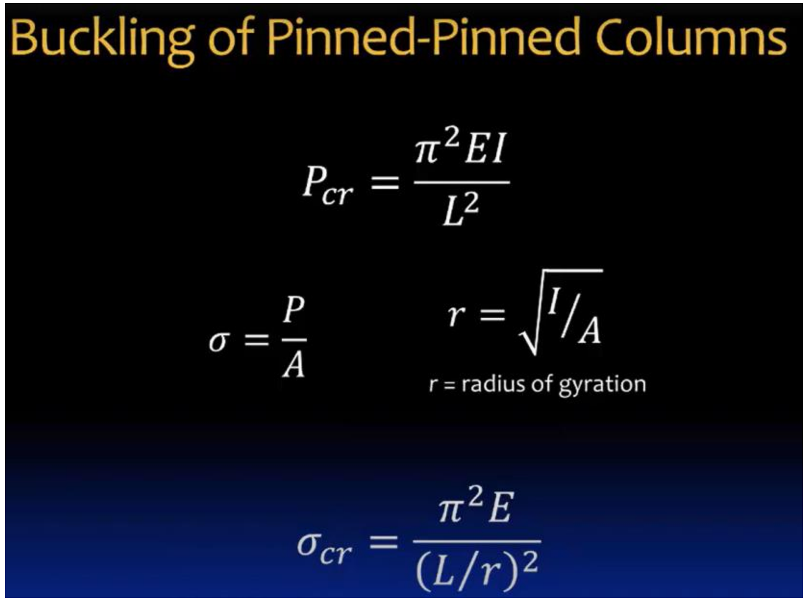
Affected and connecting elements are normally short enough and thick enough that flexural effects, if present at all, do not impact the design. When such elements are long enough and thin enough that flexural effects must be considered, the following provisions are used for determining the available strength.

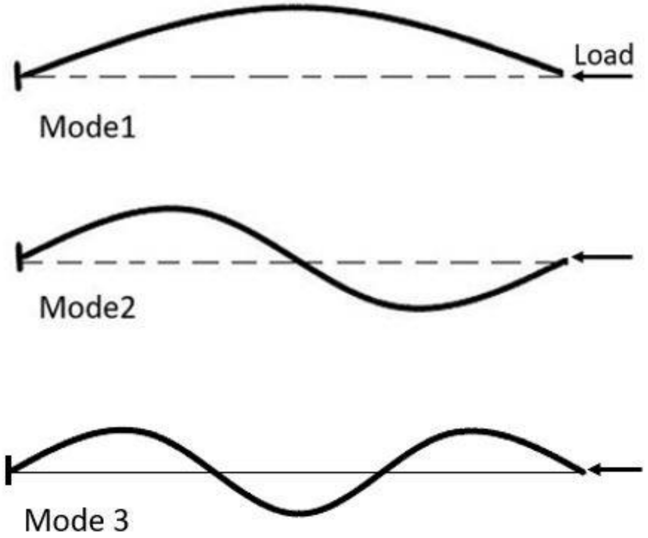
Yielding, Lateral-Torsional Buckling, and Local Buckling

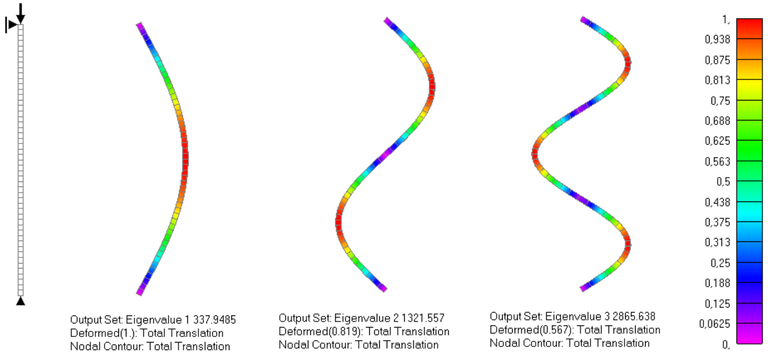
Generally, the available flexural strength, ϕM_n or M_n/Ω, which must equal or exceed the required flexural strength of affected and connecting elements, M_u or M_a, respectively, is determined in accordance with AISC *Specification* Section J4.5 and Chapter F. The Table User Note in AISC *Specification* Section F1.1 provides guidance based upon cross-section shape for the applicable Chapter F section.

Treatment of coped beams is provided in the following.

FEA Linear Buckling analysis





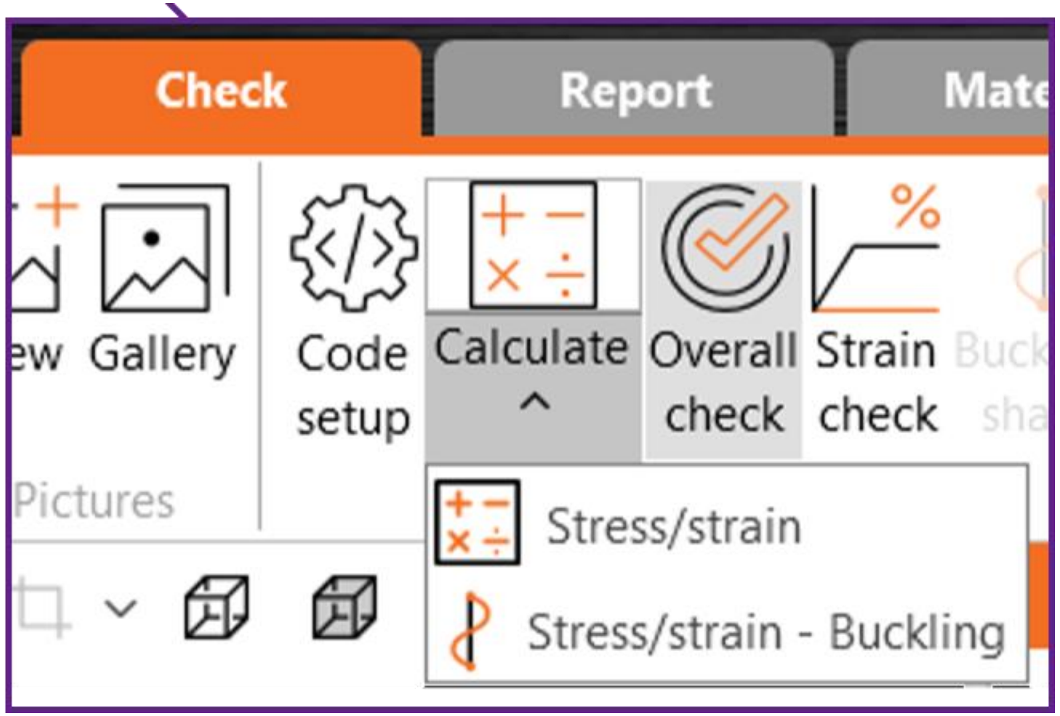


LINEAR BUCKLING ANALYSIS IN IDEA StatiCa

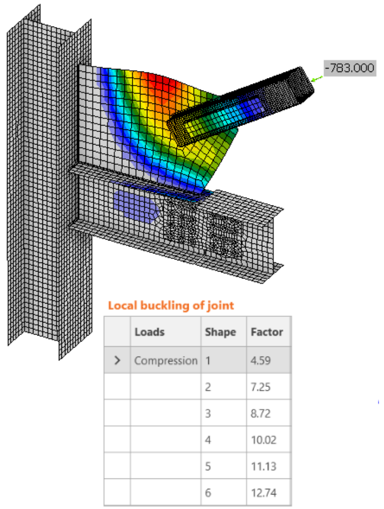
Calculate yesterday's estimates

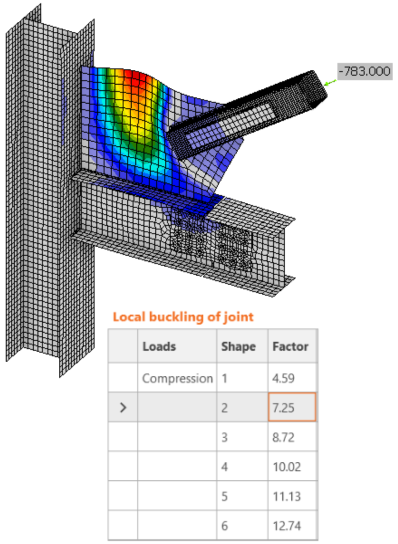
Linear buckling analysis in IDEA StatiCa

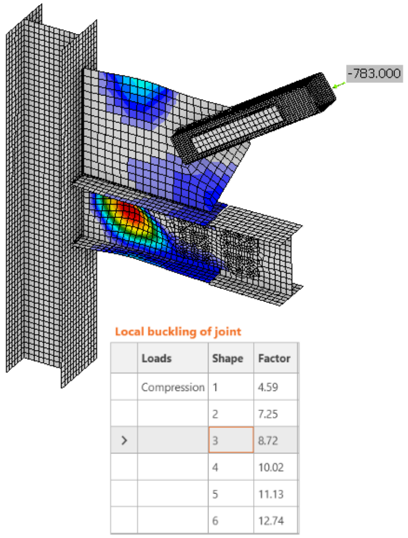
- Linear buckling analysis is performed in the whole joint
- The results are predicted in buckling mode shapes
- Critical load, at which buckling of the perfect model occurs, is calculated for each buckling mode
- Critical load is presented by multipliers of the load acting on the joint
- According to the buckling mode and critical load multiplier, the user can determine the safe buckling design

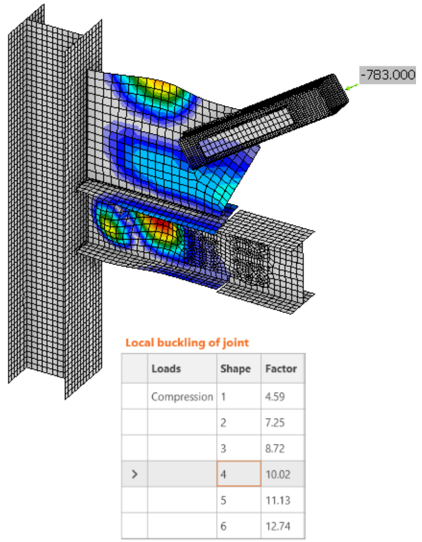


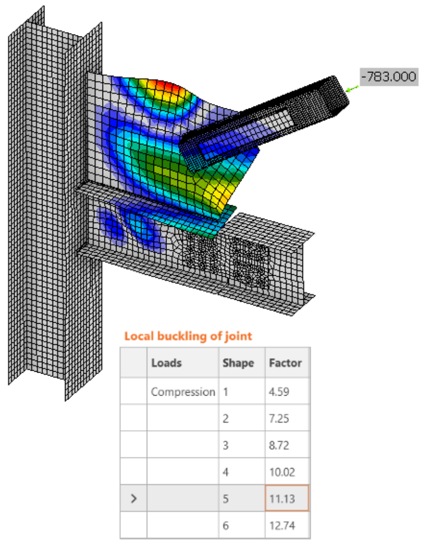
Buckling mode shapes

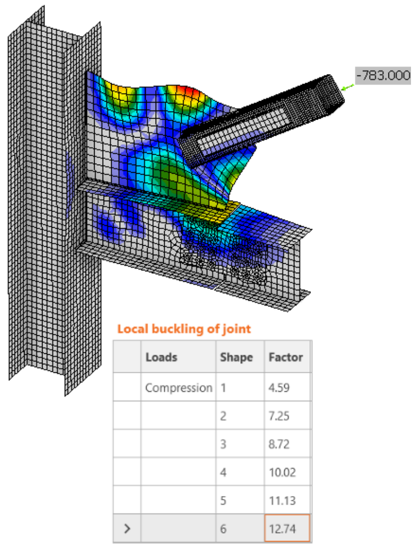












Global or local buckling?

Global

- Global buckling: affecting the stability of the joint
- Plates connecting individual members
- i.e., Gusset plates with only one side connected

Local

- Local buckling: not affecting the stability of the joint, but influencing the plates in other parts of the connection
- i.e., Member plates, haunches, stiffeners, ribs, bracket plates
- Gusset plates with more than 2 edges connected

Recommended limit factors

Global

Steel Fy	AISC Method	Critical buckling factor
36 ksi	LRFD	αcr>12.7
50 ksi	LRFD	αcr>9.16
36 ksi	ASD	αcr>21
50 ksi	ASD	αcr>15

Local

AISC Method	Critical buckling factor
LRFD	αcr>3 – Member plates αcr>4 – Connection plates (i.e. bracket plates)
ASD	αcr>4.5 – Member plates αcr>6 – Connection plates (i.e. bracket plates)

Slenderness limit ratio for non slender connecting plates

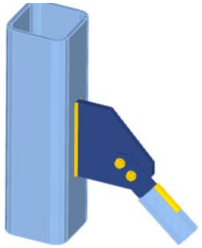
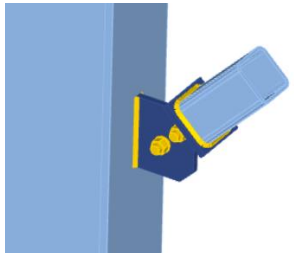
$$\frac{Lc}{r} = 25$$

$$Fe = \frac{\pi^2 E}{(Lc/r)^2} = \frac{\pi^2 (29{,}000\ ksi)}{(25)^2} = 458\text{ksi}$$

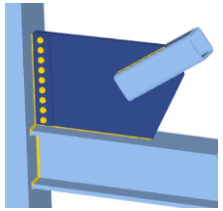
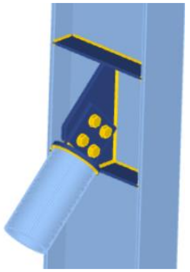
$$\alpha\text{cr} = \frac{Fe}{Fy} = \frac{458}{36} = 12.7$$

Gusset plates categories

- One side restraint – **global** limit factors

- Two/three sides restrained – **local** limit factors

- The stability of the gusset plate depends on how many edges are restrained.

IDEA StatiCa linear buckling analysis process

1. Design the connection

- Run regular stress/strain analysis and verify you comply with all the code checks

2. Run Buckling analysis

- Go to Check tab>Calculate>Buckling

3. Analyze results and decide

- Review the mode shapes and buckling load factors
- Compare against recommended factors
- Higher than recommended factor?

4. Re-iterate until Buckling is OK

- If the connecting elements are under the recommended factors, use your engineering judgement to decide the solution

What to do if the buckling factor is below the recommended limit factors?

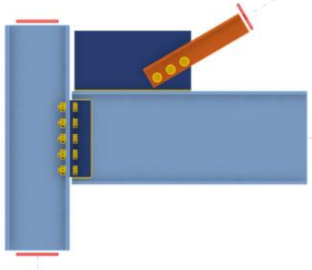
Use your engineering judgement!

If it is a global buckling issue (buckling happening in one of the connecting elements), stiffen the connection

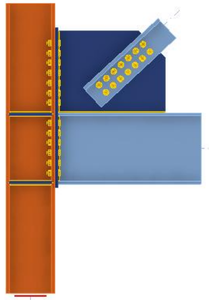
If it is a local member plates issue, review the member design (IDEA StatiCa member app)

Local buckling in thin walled/hollow section members, a non-linear buckling analysis is probably required (IDEA StatiCa member app)

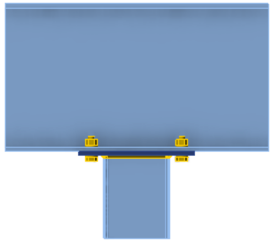
Samples



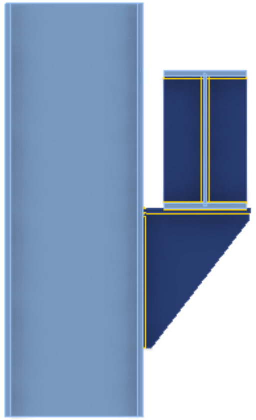
Ex. 5.4 – DG29



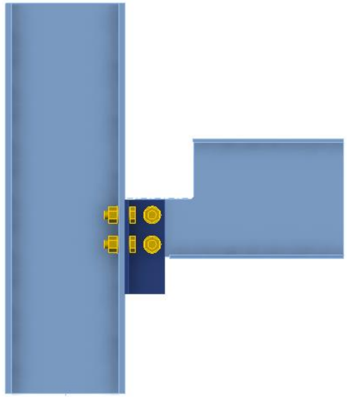
Ex. 5.1 – DG29



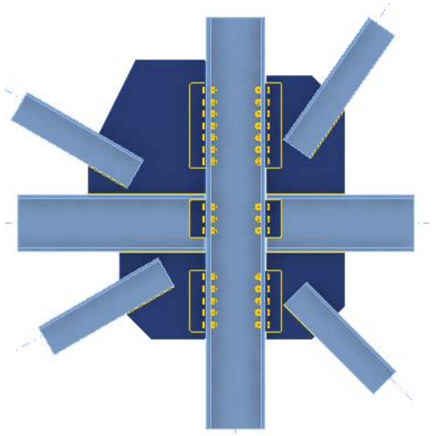
Ex. 6.1 – DG24



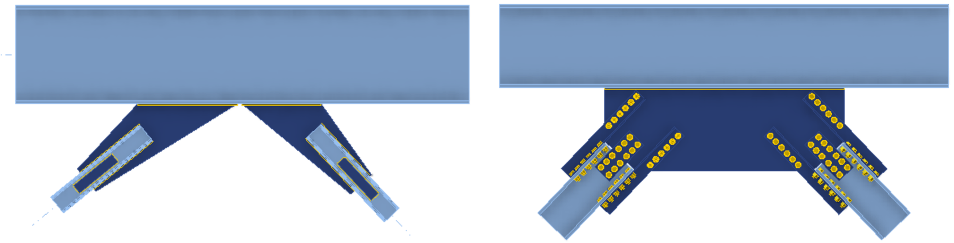
Bracket plates



Coped beam

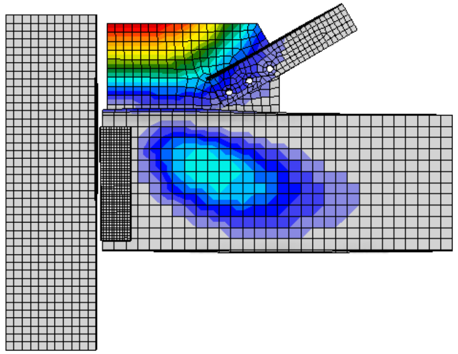


Multiple gusset plates



Chevron braces

Example 5.4 DG29



Global

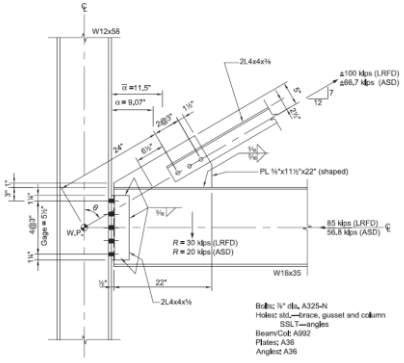


Fig. 5-10. Special Case 3—strong axis.

$= 0.90(36 \text{ ksi})(3.47 \text{ in.}^2)$ $= 112 \text{ kips} > 100 \text{ kips}$ **o.k.**	$\Omega \quad \Omega$ $= \dfrac{(36 \text{ ksi})(3.47 \text{ in.}^2)}{1.67}$ $= 74.8 \text{ kips} > 66.7 \text{ kips}$ **o.k.**

Check the gusset plate for compression buckling on the Whitmore section

The available compressive strength of the gusset plate based on the limit state of flexural buckling is determined from AISC *Specification* Section J4.4. Use an effective length factor of $K = 1.2$ due to the possibility of sidesway buckling (see AISC *Specification* Commentary Table C-A-7.1). From Figure 5-10, the unbraced length of the gusset plate is shown as 6½ in.

$$\frac{KL}{r} = \frac{1.2(6\frac{1}{2} \text{ in.})}{\frac{1}{2} \text{ in.}/\sqrt{12}} = 54.0$$

Because $KL/r > 25$, use AISC *Manual* Table 4-22 to determine the available critical stress for the 36-ksi gusset plate. Then, the available compressive strength can be determined from AISC *Specification* Sections E1 and E3:

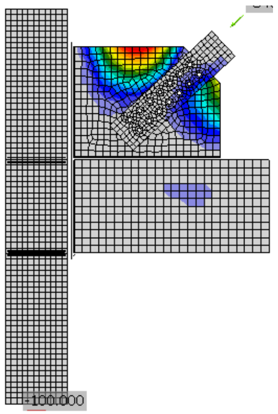
LRFD	ASD
$\phi_c F_{cr} = 27.8 \text{ ksi}$ $\phi_c P_n = \phi_c F_{cr} A_w$ $= (27.8 \text{ ksi})(3.47 \text{ in.}^2)$ $= 96.5 \text{ kips} < 100 \text{ kips}$ **n.g.**	$\dfrac{F_{cr}}{\Omega_c} = 18.5 \text{ ksi}$ $\dfrac{P_n}{\Omega_c} = \dfrac{F_{cr} A_w}{\Omega_c}$ $= (18.5 \text{ ksi})(3.47 \text{ in.}^2)$ $= 64.2 \text{ kips} < 66.7 \text{ kips}$ n.g.

Because the gusset fails in buckling, a thicker gusset could be used, or the material could be changed to ASTM A572 Grade 50. The thicker gusset will be used here.

Try a ⅝-in.-thick gusset plate:

$$\frac{KL}{r} = \frac{1.2(6\frac{1}{2} \text{ in.})}{\frac{5}{8} \text{ in.}/\sqrt{12}} = 43.2$$

Example 5.1 DG29 - Solution



Check the gusset plate for compression buckling on the Whitmore section

The available compressive strength of the gusset plate based on the limit state of flexural buckling is determined from AISC *Specification* Section J4.4, using an effective length factor, K, of 0.50 as established by full scale tests on bracing connections (Gross, 1990). Note that this K value requires the gusset to be supported on both edges. Alternatively, the effective length factor for gusset buckling could be determined according to Dowswell (2006). In this case, because KL/r is found to be less than 25 assuming $K = 0.50$, the same conclusion, that buckling does not govern, will be reached using either method.

$$r = \frac{t_g}{\sqrt{12}}$$
$$= \frac{1.00 \text{ in.}}{\sqrt{12}}$$
$$= 0.289 \text{ in.}$$

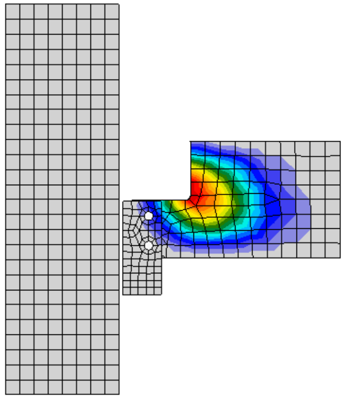
From Figure 5-2, the gusset plate unbraced length along the axis of the brace has been determined graphically to be 9.76 in.

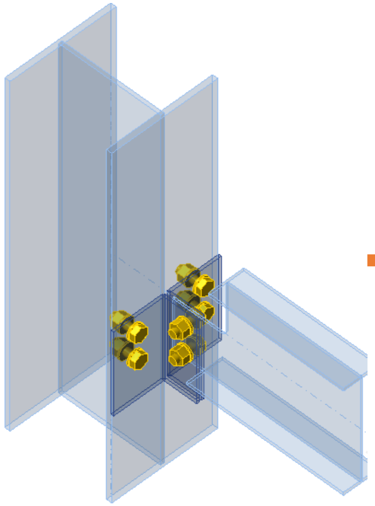
$$\frac{KL}{r} = \frac{0.50(9.76 \text{ in.})}{0.289 \text{ in.}}$$
$$= 16.9$$

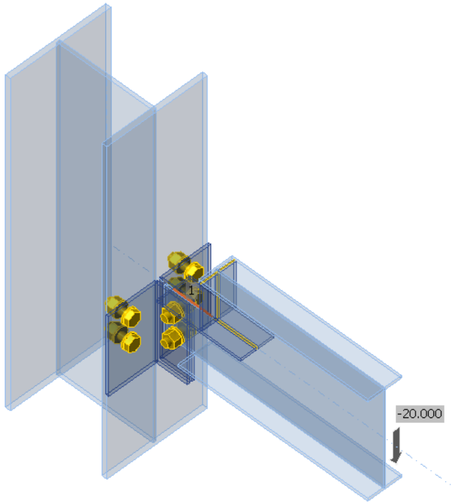
Because $\frac{KL}{r} < 25$, AISC *Specification* Equation J4-6 is applicable, and the available compressive strength is:

LRFD	ASD
$\phi P_n = \phi F_y A_g$ $= 0.90(50 \text{ ksi})(20.9 \text{ in.}^2)$ $= 941 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\frac{P_n}{\Omega} = \frac{F_y A_g}{\Omega}$ $= \frac{(50 \text{ ksi})(20.9 \text{ in.}^2)}{1.67}$ $= 626 \text{ kips} > 560 \text{ kips}$ **o.k.**

Coped beam – Solution







Web Reinforcement of Coped Beams

When the strength of a coped beam is inadequate, either a different beam with a thicker web can be selected to eliminate the need for reinforcement, or reinforcement can be provided to increase the strength. In spite of the increase in material cost, the former solution may be the most economical option due to the appreciable labor cost associated with adding stiffeners and/or doubler plates. When the latter solution is required, some typical reinforcing details are illustrated in Figure 9-11.

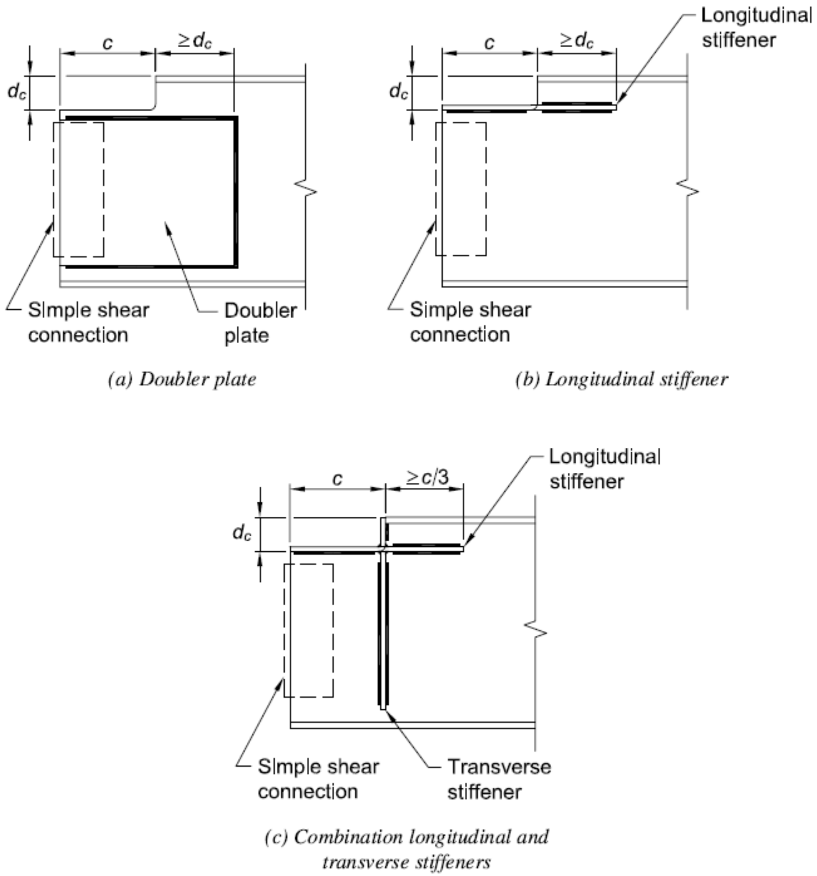


Fig. 9-11. Web reinforcement of coped beams.

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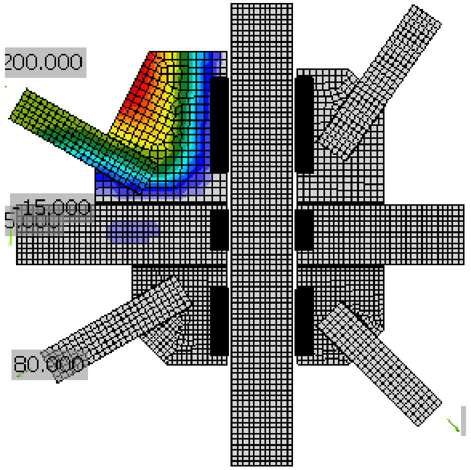
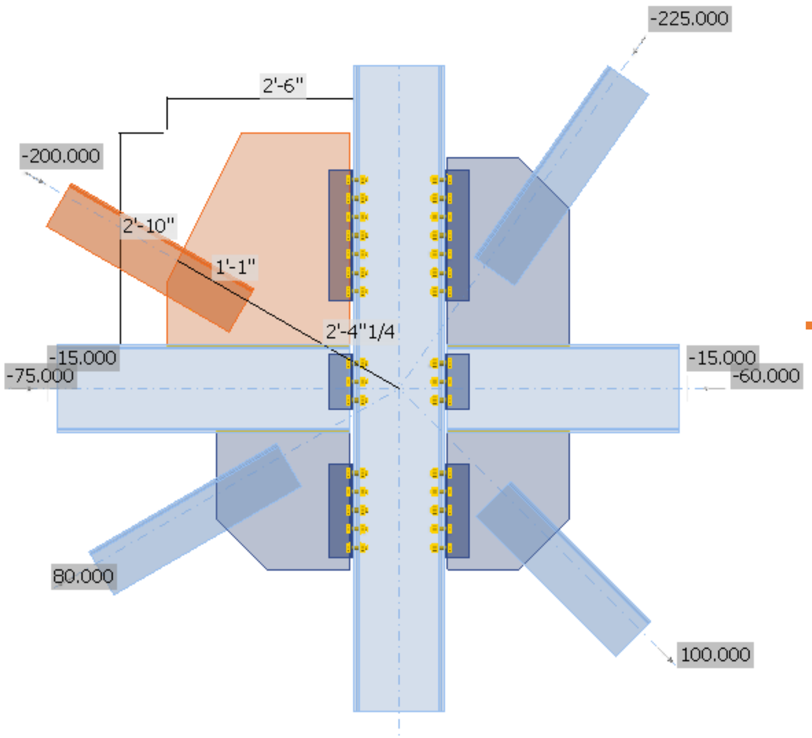
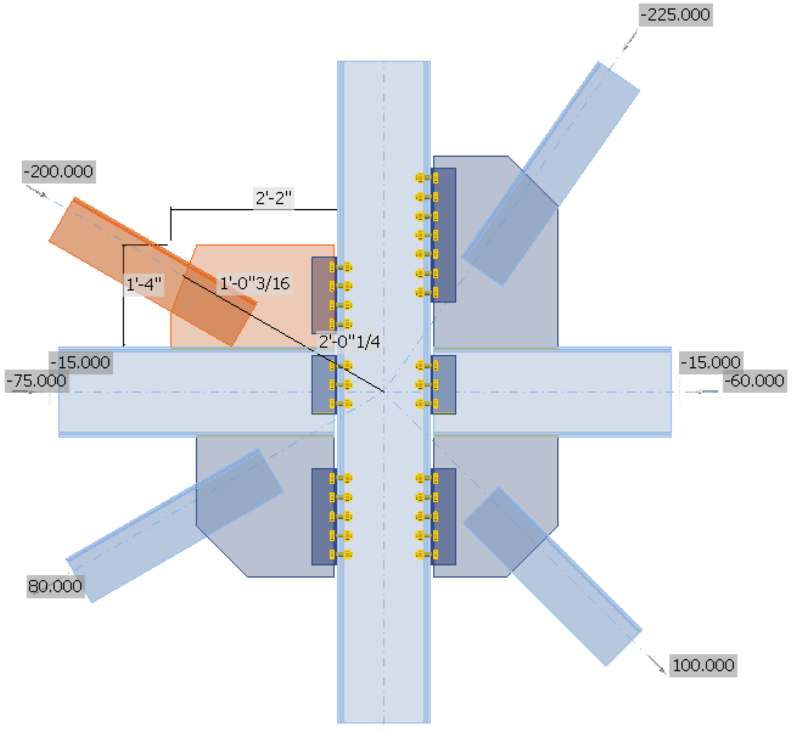
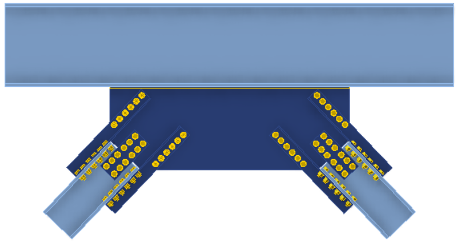
Multiple gusset plates – Solution

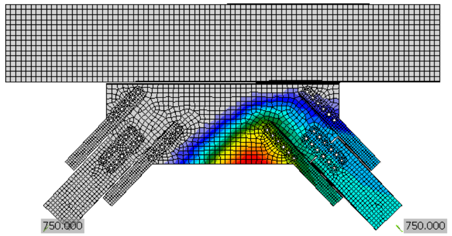




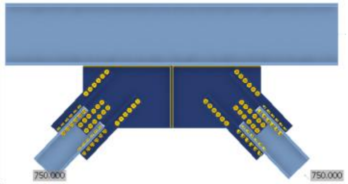


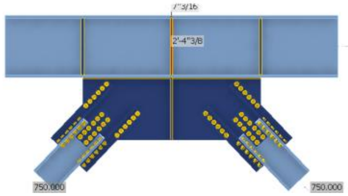
Chevron braces – Claw angle connection





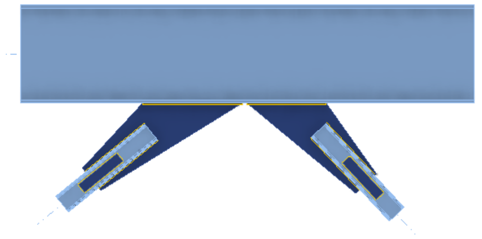
Even though the gusset plate is connected only to one side, the plate is restrained in two sides. One is the connection to the beam, the second is the restriction from the other brace in tension, that is why we can consider that buckling issue as a local category.





Increasing the thickness wasn't enough, stiffeners in the plate helped.
Then a second test with stiffeners in the beam helped more.

Chevron braces



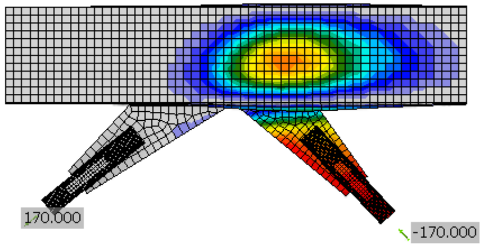
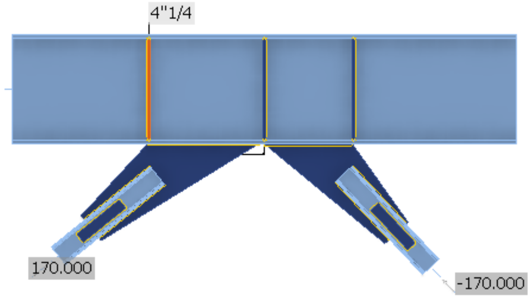
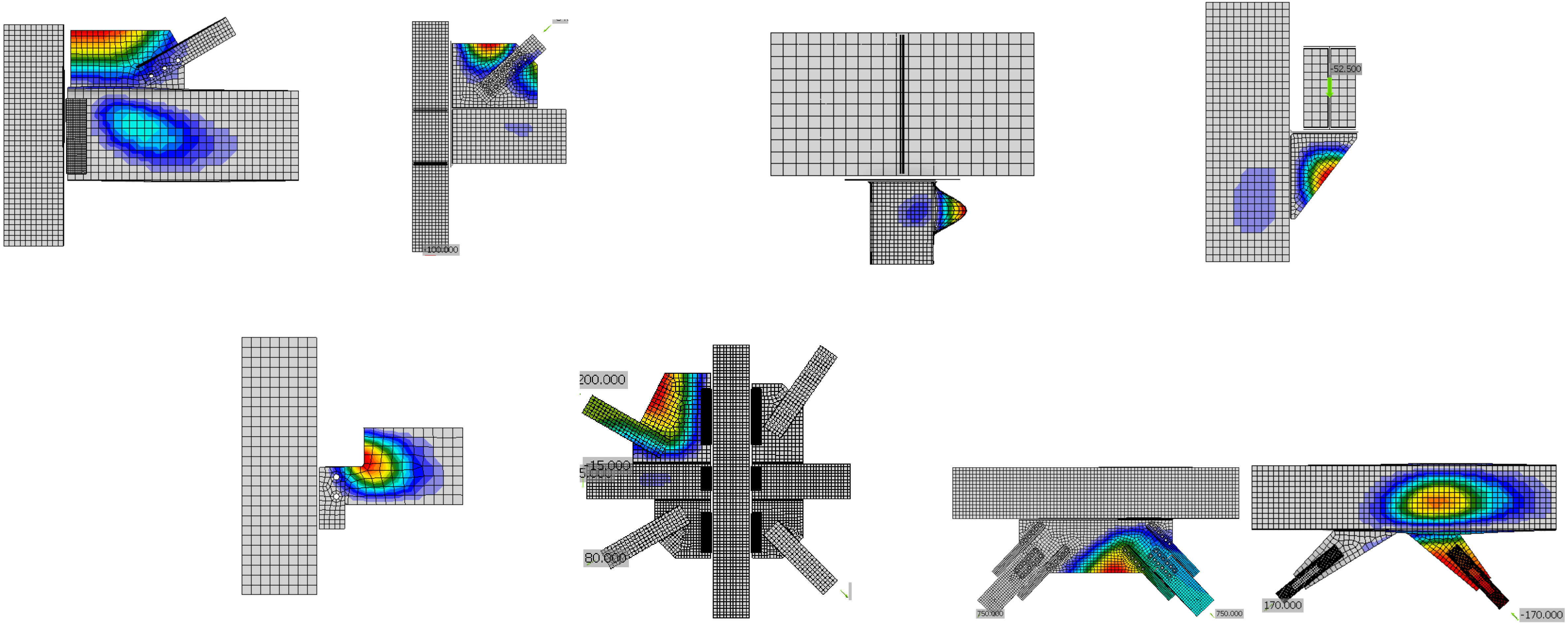


Plate is restrained at one side -
Global buckling issue

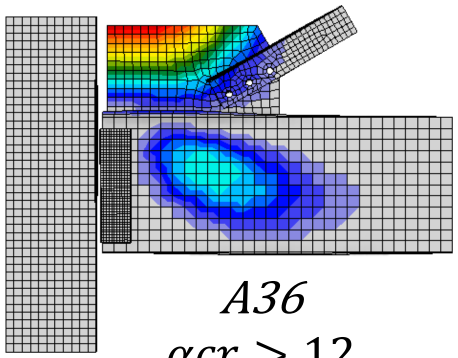


Increasing the thickness wasn't enough, stiffeners at the beam were needed to improve the stockiness of the connection

Global or local?



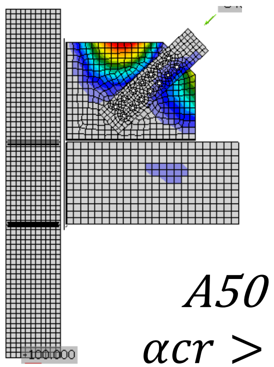
Global or local?



$A36$

$\alpha cr > 12$

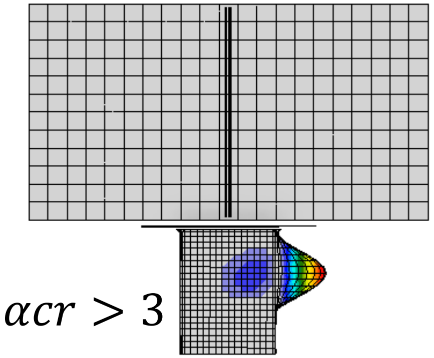
Global- One side connected



$A50$

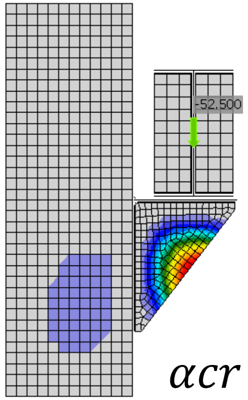
$\alpha cr > 4$

Local- 2 sides connected



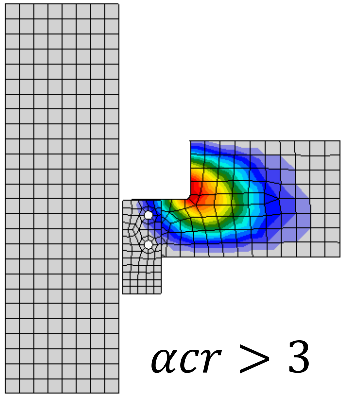
$\alpha cr > 3$

Local- Member plate



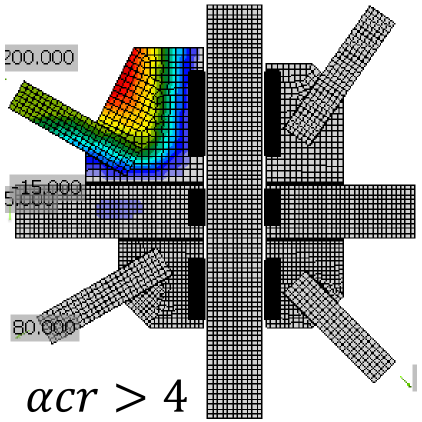
$\alpha cr > 4$

Local- Bracket plates



$\alpha cr > 3$

Local – Member plate

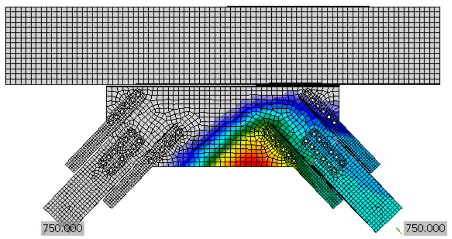


$\alpha cr > 4$

Local – 2 Sides connected

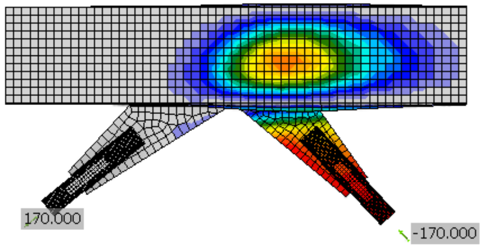
Local- 2 sides connected

$\alpha cr > 4$



Global – one side

$A572\ \alpha cr > 9$



SUMMARY

Calculate yesterday's estimates

- IDEA StatiCa connection uses linear buckling analysis to provide a buckling factor only and recommended limits 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

Resources

- Catalog of AISC limit states and design requirements
- Buckling analysis per AISC in IDEA StatiCa
- Stability of Bracket Plates using Local Buckling Analysis and Material Nonlinear Analysis (AISC)
- Chevron Brace Connection in a braced frame (AISC)
- Coped beams Verifications
- IDEA StatiCa for buckling analysis in member design

Q&A

Upcoming events

In person

- SEA of North Carolina - 8/15-16
- SEA of Alabama – 8/29
- SEA of California – 9/3-6
- SEA of Ohio – 9/12-13
- SDS2 User Summit – 9/25-26
- SEA of Colorado – 10/3
- NCSEA Summit – 11/6-8

Webinars

- User success story – 7/31
- TBD – 8/26
- TBD – 9/25
- Updates from V24.1 -10/30
- TBD – 11/27