1. (3 / 25) Given the following cross-section, identify three ways to increase the block shear capacity of the connection.

- Increase yield & U.L.T. capacity of the steel
- Increase thickness of the channels
- Increase thickness of the gusset plate

2. (3 / 25) Which net sections must be investigated as part of net section fracture in the following connection? (i.e., the net sections that you cannot immediately rule out without having dimensions and actually performing the calculations.) Points will be deducted for listing sections that do not actually need to be investigated. The number of points assigned to this problem has no correlation to the number of sections you should be identifying...

- AB
- AC B
3. (2 /25) What is the purpose behind the AISC requirement that 
$a \leq r \left( \frac{3}{4} \right) \left( \frac{K_f}{r} \right)_{\text{max}}$?

To limit connector spacing in built-up columns such that the column buckles as a whole, rather than individual components buckling.

4. (2 /25) In what case(s) can the stiffness reduction factor, $\tau_s$, NOT be used?

When column buckling is elastic.

5. (3 /25) Given the following built-up cross-section loaded in uniform compression, provide a bulleted list of all checks that should be performed to ensure that the chosen member is adequate to support a set of given loads. (Points will be deducted for listings that do not actually need to be investigated.) The number of points assigned to this problem has no correlation to the number of checks you should be identifying...

- Local Buckling
- Flexural Buckling
- Flexural - Torsional Buckling
6. (3 / 25) Describe the phenomenon of shear lag, and in what situation(s) it must be accounted for.

Shear lag occurs when one or more elements are not connected to the connecting element (e.g., gusset plate), and must be accounted for in that scenario. It is the phenomenon in which stress transfer occurs over a length of the cross-section, rather than directly at the point of connection. The resistance of the entire cross-section is not utilized at the point of connection by the unconnected element(s).

7. (4 / 25) Draw a stress-strain diagram typical of mild or HSLA steel, and label the following:
- Modulus of elasticity, \( E \)
- Yield strength
- Ultimate strength
- Yield plateau
- Strain hardening modulus, \( G \)
- Elastic region
- Strain hardening region
- Necking region
8. (5 / 25) Describe the fundamental difference between ASD and LRFD approaches to design.

LRFD
\[ \gamma Q \leq \Phi R_n \]

LRFD incorporates both load and resistance factors, which separately account for uncertainties in loads and member resistances.

ASD
\[ f_{all} = \frac{F_{lim}}{\Omega} \]

ASD utilizes a factor of safety, which accounts for uncertainties in both load and resistance.

(2 pts.) Bonus: Who is the “Father of LRFD”?  
TED GALAMBOS
1. [25 pts / 100 pts] The built-up cross-section detailed below is planned for use as column EF in the steel-framed building shown. The columns are braced such that strong axis buckling will control. The column is to be constructed of Grade A572-50 steel. What is the design capacity of the column? Use the LRFD design approach, and perform all necessary checks. Show ALL of your work to obtain full credit, and provide reference to the AISC Manual / Specification when you use an equation, table, or chart.
Cross-Section Properties:

\[ A = 3 \left[ \frac{16}{12} \times \frac{1}{2} \right] + 2 \left[ 3 \times 1 \right] = 40 \text{ in}^2 \]

\[ I_x = 3 \left[ \frac{1}{12} (\frac{1}{2})(12) + \frac{d^2}{4} \right] + 2 \left[ \frac{1}{12} (3)(1)^2 + (3)(8.5)^2 \right] \\
= 3 \left[ 140.697 \right] + 2 \left[ 0.647 + 578 \right] \\
= 1,069.33 \text{ in}^4 \]

\[ Y_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{1,069.33}{40}} = 16.46 \text{ in} \]

Determine Effective Length of Column:

\[ G_t = \frac{Z (T/L)}{Z (T/L)_{col}} \quad \text{(Comm. C2)} \]

\[ G_{tf} = \frac{(1069.33/28)}{(2100/82)} = 0.908 \]

\[ G_{te} = \frac{(1069.33/28) + (1069.33/28)}{(2100/82)} = 2.18 \]

Using the Alignment Chart for Braced Frames (Fig. C-2.2, pg. 16.1-241),

\[ K = 0.82 \]

\[ (KL)_{x,y} = 0.82(28') = 22.96' \]

Calculate Column Capacity:

\[ \left( \frac{KL}{T} \right)_{x} = \left( \frac{22.96' \times 12 \times 12}{80.46'} \right) = 42.65 \]

\[ 4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,000}{50}} = 119.43 \]

\[ F_{cr} = (0.658) (F_y) (F_t) \]

where \( F_y = \frac{T^2}{(42.65)^2} = 157.35 \)

\[ F_{cr} = (0.658)(50/157.35)(50 \text{ ksi}) = 43.77 \text{ ksi} \]
\[ P_n = 0.90 (1750.75 \text{ kips}) = 1575.03 \text{ kips} \]

**CHECK LOCAL BUCKLING**

All elements in the built-up cross-section are stiffened.

Case 2 of Table B4.1 (p. 181-18) applies. Choose the element with the largest slenderness (one of the webs).

\[ \lambda = \frac{b}{\frac{t}{2}} = \frac{16}{\frac{1}{2}} = 32 \]

\[ \lambda_y = 1.49 \sqrt{\frac{E}{F_y}} = 1.49 \sqrt{29,000,000 \text{ ksi}} = 35.88 \]

Since \( \lambda < \lambda_y \), local buckling will not control.

**Conclusion**: The column capacity is 1576 kips.
2. [25 pts / 100 pts] A column is braced as shown in the figure below, and is subject to factored axial loads of 100kip dead load and 200kip live load. The column is located in a braced steel frame (sideways inhibited). You may assume that all connections are approximately pinned for the purpose of design. Choose the lightest W14 shape to resist the loads. Use A36 steel. Use the LRFD design approach, and perform all necessary design checks. Show ALL of your work to obtain full credit, and provide reference to the AISC Manual / Specification when you use an equation, table, or chart.

Trial #1

- Assume $\phi F_{cr} = 0.7 (F_y) = 0.7 (36,000) = 25.2 \text{kips}$

Required area: $A_g \geq \frac{200 \text{kips}}{25.2 \text{kips}} = 7.90 \text{ in}^2$

Try a W14 x 38 (Table 1-1, p. 1-22 of Manual).

Solve for capacity of a W14 x 38

- $A_g = 12.16 \text{ in}^2$
- $T_x = 5.82 \text{ in}$
- $r_y = 1.89 \text{ in}$
- **Buckling Capacity (W14x36)**

  \[
  \left( \frac{KL}{T} \right)_x = \left( \frac{1.35'12''}{5.89''} \right) = 61.86
  \]

  \[
  \left( \frac{KL}{T} \right)_y = \left( \frac{1.15'12''}{1.89''} \right) = 95.24 \quad \text{Weak Axis Buckling Controls.}
  \]

  \[
  4.71 \sqrt{\frac{E^2}{F_y}} = 4.71 \sqrt{\frac{29,000}{20}} = 133.68
  \]

  Inelastic Buckling Governs.

  \[
  F_{cr} = 0.658 \left( \frac{T_f}{F_y} \right) (F_y)
  \]

  Where:

  \[
  F_e = \frac{\pi^2 E}{(KL/T)^2} = \frac{\pi^2 (29,000)}{(95.24)^2} = 21.56 \text{ kips}
  \]

  \[
  F_{cr} = 0.658 \left( \frac{22.25 \text{kips}}{36 \times 61} \right) = 22.23 \times 61
  \]

  \[
  P_u = (22.23 \text{kips})(12.4 \text{ in})^2 = 281.34 \text{ kips}
  \]

  \[
  \Phi P_u = 0.90 \left( 281.34 \text{ kips} \right) = 253.22 \text{ kips}
  \]

  \[
  \Phi P_u < P_u \quad \text{N.G. - Select the next larger shape, a W14x48.}
  \]

**Solution for Capacity of a W14x48:**

\[
A_g = 14.1 \text{ in}^2
\]

\[
r_x = 5.38 \text{ in}
\]

\[
r_y = 1.31 \text{ in}
\]

- **Buckling Capacity (W14x48):**

  \[
  \left( \frac{KL}{T} \right)_x = \left( \frac{30'12''}{5.85} \right) = 61.54
  \]

  \[
  \left( \frac{KL}{T} \right)_y = \left( \frac{15'12''}{1.91} \right) = 94.24 \quad \text{Weak Axis Controls}
  \]

  \[
  \left( \frac{KL}{T} \right)_{max} = 4.71 \sqrt{\frac{E}{F_y}} = \text{Inelastic Buckling Governs}
  \]

  \[
  F_{cr} = 0.658 \left( \frac{F_y}{F_e} \right) (F_y)
  \]

  Where:

  \[
  F_e = \frac{\pi^2 E}{(KL/T)^2} = \frac{\pi^2 (29,000)}{(94.24)^2} = 32.22
  \]
\[ F_{cr} = (0.1558 \times (36/32.22)) (36 \text{ ksi}) = 22.56 \text{ ksi} \]

\[ P_n = (22.56 \text{ ksi}) (14.1 \text{ in}^2) = 318.03 \text{ kips} \]
\[ \Phi P_n = 0.9 (318.03 \text{ kips}) = 286.23 \text{ kips} \]
\[ \Phi P_n < P_n \implies \text{N.G.} \implies \text{select the next larger, a W14 \times 53.} \]

**Solve for Capacity of a W14 \times 53:**

- **A_y = 15.6 \text{ in}^2**
- **I_x = 5.89 \text{ in}^2**
- **Q_3 = 1.92 \text{ in}**

**Flexural Buckling Capacity (W14 \times 53)**

\[ \left( \frac{KL}{r} \right)_x = \left( \frac{20.12}{5.89} \right) = 3.39 \]
\[ \left( \frac{KL}{r} \right)_y = \left( \frac{15.12}{1.92} \right) = 7.93 \quad \text{WEEK AXIS CONTROLS.} \]
\[ \left( \frac{KL}{r} \right)_{max} = 4.71 \sqrt{\frac{E}{F_y}} \quad \text{ELASTIC BUCKLING GOVERNS} \]

\[ F_{cr} = (0.1558 \times \left( \frac{F_y}{F_k} \right) \times (F_y) \]

\[ \text{where: } F_y = \frac{\pi^2 (29000)}{(73.75)^2} \]

\[ F_{cr} = (0.1558 \times (21.5/53.5)) (36 \text{ ksi}) = 22.57 \text{ ksi} \]
\[ P_n = (22.57 \text{ ksi}) (15.6 \text{ in}^2) = 353.58 \text{ kips} \]
\[ \Phi P_n = 0.9 (353.58 \text{ kips}) = 318.22 \text{ kips} \]
\[ \Phi P_n > P_n \implies \text{A W14 \times 53 is sufficient in flexural buckling.} \]

**Check Local Buckling (W14 \times 53)**

\[ \lambda_s = 60.11 \quad \text{vs.} \quad \lambda_r = 0.54 \sqrt{\frac{E}{F_y}} = 15.89 \quad \text{O.K.} \]
\[ \lambda_{web} = 30.9 \quad \text{vs.} \quad \lambda_r = 1.49 \sqrt{\frac{E}{F_y}} = 42.29 \quad \text{O.K.} \]

The section will not fail in local buckling.
Conclusion: A W14 x 53 is the lightest sufficient section to support the loads.

Use a W14 x 53 made of A36 steel.
3. [25 pts / 100 pts] Determine the maximum service load, $P_{\text{service}}$, the tension member shown can safely support using LRFD. The member is comprised of two A572-50 L7x4x3/8, connected LLBB. The member is connected to the gusset plate with eight 3/8" diameter bolts. There are three holes that the fabricator drilled for 3/8" bolts on the top angle leg, but the contractor forgot to attach them to anything; those three holes sit empty in the constructed member. In addition to all other required checks, investigate at least one potentially-controlling block shear failure mode in your analysis.

You may assume that the total load is 30% dead load and 70% live load and that the controlling load combination is $P_u = 1.2(\text{DL}) + 1.6(\text{LL})$

Show ALL of your work to obtain full credit, and provide reference to the AISC Manual / Specification when you use an equation, table, or chart.

**SHAPE - 2L 7 x 4 x 3/8**

$A_g = 6.00 \text{ in}^2$ (Table 1-15, p. 1-100)

**GROSS SECTION YIELDING**

$P_{n} = f_{y} A_g = 50 \times 3/8 \times 6.00 = 300 \text{ kips}$

$\phi P_{n} = 0.90 (300 \text{ kips}) = 270 \text{ kips}$

**NET SECTION FRACTURE**

- Flatten each angle out to account for the staggered holes.
- Subtract out the angle thickness so it is not double accounted for.
\[ P_n = F_u A_e \]

where \( A_e = u A_{net} \)

where \( u = 1 - \frac{h}{L} \)

(Case 2, Table D3.1 - p. 16.1-29)

\[ u = 1 - \frac{0.861}{12} = 0.928 \]

**NET SECTION AB:**

\[ A_{net} = A_g - A_{holes} + \frac{L}{4g} (t) \]

\[ = 8 \text{ in}^2 - 2(3/4 - 1/8)(2.3/8) + 0 \]

\[ = 8 \text{ in}^2 - 1.3125 \text{ in}^2 \]

\[ = 6.6875 \text{ in}^2 \]

**NET SECTION ABC:**

\[ A_{net} = A_g - A_{holes} + \frac{L}{4g} (t) \]

\[ = 8 \text{ in}^2 - 3(3/4 + 1/8)(2.3/8) + \frac{(2)^2}{4(4.625)} (2.3/8) \]

\[ = 8 \text{ in}^2 - 1.96875 \text{ in}^2 + 0.119216 \text{ in}^2 \]

\[ = 6.1934 \text{ in}^2 \]

**CONTROLLING SECTION:**

\[ P_n = (6.5^{0.5})(6.1934) = 373.59 \text{ kips} \]

\[ \phi P_n = 0.75(373.59) = 280.19 \text{ kips} \]

**CONTROLS OVER GSY.**
Block shear: Block shear of the angles will control over the gusset plate since they are the same grade of steel & the gusset plate is thicker than the two angles.

Failure mode (c)

\[ R_n = 0.6 \cdot Fu_{\text{Any}} + U_{\text{bs}} \cdot Fu_{\text{Ant}} \leq 0.6 \cdot Fy_{\text{Agv}} + U_{\text{bs}} \cdot Fu_{\text{Ant}} \]

- Gross area in tension: \( A_{Gt} = (4'' \times \frac{3}{8}'')(2) = 3 \text{ in}^2 \)
- Net area in tension: \( A_{nt} = A_{Gt} - \text{holes} = 3 \text{ in}^2 - 1.5 \left( \frac{3}{8}'' + \frac{1}{10}'' \right) \left( \frac{3}{8}'' \right) \approx 2.086 \text{ in}^2 \)
- Gross area in shear, Agv: \( A_{Agv} = 14'' \left( \frac{3}{8}'' \right) \approx 10.5 \text{ in}^2 \)
- Net area in shear, Ant: \( A_{nt} = A_{Agv} - \text{holes} = 10.5 \text{ in}^2 - 2.5 \left( \frac{3}{8}'' + \frac{1}{10}'' \right) \left( \frac{3}{8}'' \right) \approx 8.347 \text{ in}^2 \)

\( U_{\text{bs}} = 1.0 \) because tension stress is uniform.

\[ 0.6 \cdot Fu_{\text{Any}} = 0.6 \left( 65 \text{ ksi} \right) \left( 8.347 \text{ in}^2 \right) = 326.32 \text{ ksi} \]

\[ U_{\text{bs}} \cdot Fu_{\text{Ant}} = 1.0 \left( 65 \text{ ksi} \right) \left( 2.086 \text{ in}^2 \right) = 135.59 \text{ ksi} \]

\[ 0.6 \cdot Fy_{\text{Agv}} = 0.6 \left( 50 \text{ ksi} \right) \left( 10.5 \text{ in}^2 \right) = 315 \text{ ksi} \]

\[ R_n = 326.32 + 135.59 = 361.91 \text{ ksi} \]

\[ R_n = 950.59 \text{ ksi} \]

\[ \phi R_n = 0.75 \left( 950.59 \text{ ksi} \right) = 337.94 \text{ ksi} \]
**Failure Mode (b):**

- \(A_{gt} = (2\text{"})(3/16\text{"})(2\times) = 1.5\text{ in}^2\)
- \(A_{nt} = 1.5\text{ in}^2 - 2(2\times)(3/4\times)(1/8\times) = 0.891 \text{ in}^2\)
- \(A_{gt} = 2[(4\times)(1/8\times)] = 2\text{ in}^2\)
- \(A_{nv} = 21.43^2 - 7(3/4\times)(1/8\times) = 10.73 \text{ in}^2\)

0.6 \(F_u A_{nv} = 0.6(65 \times 10^6)(10.73 \text{ in}^2) = 652,646 \text{ kips}\)

0.6 \(F_u A_{nt} = 1.0(65 \times 10^6)(0.891 \text{ in}^2) = 58.72 \text{ kips}\)

0.6 \(F_y A_{gt} = 0.6(50 \times 10^6)(2\text{ in}^2) = 600 \text{ kips}\)

\(R_n = 652.646 \text{ kips} + 58.72 \text{ kips} = 680 \text{ kips} + 58.92 \text{ kips}\)

\(R_n = 687.92 \text{ kips}\)

\(\phi R_n = 0.75(687.92 \text{ kips}) = 515.9 \text{ kips}\)

**Service Loads**

\(P_u = 1.2 P_L + 1.6 P_L = 1.2(0.3P_3) + 1.6(0.7P_5) = 189.3 \text{ kips}\)

**Conclusion:** Net section fracture governs with a design strength, \(\phi R_n = 280 \text{ kips}\). The total service load that can be safely supported is 189.3 kips.