1. Describe why there are two different alignment charts - one for braced frames, and one for unbraced frames. Be specific – for example, could we use one “combined” alignment chart that covers both types of frames?

There are two different alignment charts because $k$ will always be equal to or less than 1.0 for a braced frame, and $k$ will always be greater than 1.0 for an unbraced frame.

2. Given the following square HSS section loaded in tension and connected to another member through gusset plates on its top and bottom surfaces - will shear lag be a consideration in the analysis of this member? Why or why not?

Yes, shear lag should be considered. The sides of the HSS tension member are not included in the connection, and therefore will experience uneven loading at the connection.

3. Why is the 0.2% offset method used for determining the yield point from a stress-strain diagram resulting from a coupon tension test?

The 0.2% offset method is used for determine the yield point because it is a repeatable, objective measure of the yield strength. Researcher A and Researcher B should obtain the same result for $F_y$ if using this method. Also, for some high strength steels, there is not a well-defined yield point. For these stress-strain relationships, a method is needed to quantify the yield strength of the material in a repeatable manner.
4. [2 pts / 25] True or False: The columns in the following frame will each have a $k$ value greater than 1.0.

5. [5 pts / 25] Given the channel section loaded in uniform compression with axes labeled as shown, list all checks that should be performed, including what axis/axes they should be checked about (if applicable).

   Flexural buckling about $y$, flex-tors buckling about $x$, and local buckling.

6. [3 pts / 25] Which, if any, of the following are not valid block shear rupture failure modes for a channel attached to a gusset plate as shown?

   C and D are not valid block shear failure modes because a number of bolts are left connecting the tension member to the gusset plate.
1. [40 pts/ 7 5] Consider the 20-ft long tension member shown. The A992 tension member is a 9” x 9/16” plate connected to a supporting member via two A36 gusset plates with 7/8” diameter bolts. Determine the strength of the tension member, considering block shear capacity of the connection (it is your job to determine if block shear is governed by either the tension member or the gusset plate) in addition to any other potential failure modes. Your capacity check should also consider serviceability. Consider the gusset plate width dimension (not given) to be much greater than its length.

Gross Section Yielding of the Tension Member:

\[
R_n = F_y A_g = \left( 50 ksi \right) \times \left( 5.0625 in^2 \right) = 253.13 kips
\]

\[
\phi R_n = (0.90) \left( 253.13 kips \right) = 227.81 kips
\]

Net Section Fracture of the Tension Member:

\[
R_n = F_u A_i
\]
\[ A_e = UA_{net} \]

\[ U = 1.0 \quad \text{for flat bars and plates} \]

**Net Section ACB:**

\[ A_{net} = \left( 5.0625 \text{in}^2 \right) - (3 \text{holes}) (0.5625 \text{" thick}) \left( \frac{7}{8} \text{"} + \frac{1}{16} \text{"} + \frac{1}{16} \text{"} \right) + (2) (\frac{2}{4})^2 = 4.375 \text{in}^2 \]

**Net Section AB:**

\[ A_{net} = \left( 5.0625 \text{in}^2 \right) - (2 \text{ holes}) (0.5625 \text{" thick}) \left( \frac{7}{8} \text{"} + \frac{1}{16} \text{"} + \frac{1}{16} \text{"} \right) = 3.938 \text{in}^2 \]

\[ A_e = (1.0) (3.938 \text{in}^2) = 3.938 \text{in}^2 \]

\[ R_u = (65 \text{kips}) (3.938 \text{in}^2) = 255.97 \text{kips} \]

\[ \phi R_u = (0.75) (255.97 \text{kips}) = 191.98 \text{kips} \]

**Block Shear Failure of the Tension Member:**

**Tension Member Block Shear Mode 1:**

\[ R_u = 0.6 F_m A_m + U_{hs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{hs} F_u A_{nt} \]

**Gross area in tension, \( A_{gt} \)**

\[ A_{gt} = (4\text{"}) (0.5625\text{" thick}) = 2.25 \text{in}^2 \]

**Net area subject to tension, \( A_{nt} \)**

\[ A_{nt} = A_{gt} - A_{holes} \]

\[ A_{nt} = 2.25 \text{in}^2 - \left( \frac{1}{2} + \frac{1}{3} \right) \left( \frac{7}{8} \text{"} + \frac{1}{16} \text{"} + \frac{1}{16} \text{"} \right) (0.5625\text{"}) = 1.6875 \text{in}^2 \]

**Gross area in shear, \( A_{gv} \)**

\[ A_{gv} = 2 (16.5\text{"}) (0.5625\text{"}) = 18.563 \text{in}^2 \]
Net area subject to shear, $A_{nv}$

$$A_{nv} = A_{gv} - A_{holes}$$

$$A_{nv} = 18.563^{in^2} - 2\left(1 + 1 + \frac{1}{4}\right)\left(\frac{2}{8} + \frac{1}{10} + \frac{1}{10}\right)(0.5625'^{in}) = 14.625^{in^2}$$

$$R_n = 0.6F_yA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

$$U_{bs} = 1.0 \text{ as per Commentary to Ch. J, pg. 16.1-352}$$

$$0.6F_yA_{nv} = 0.6\left(65^{ksi}\right)\left(14.625^{in^2}\right) = 570.375^{kip}$$

$$U_{bs}F_uA_{nt} = 1.0\left(65^{ksi}\right)\left(1.6875^{in^2}\right) = 109.69^{kip}$$

$$0.6F_yA_{gv} = 0.6\left(50^{ksi}\right)\left(18.563^{in^2}\right) = 556.875^{kip}$$

$$0.6F_yA_{nv} + U_{bs}F_uA_{nt} = 570.375^{kip} + 109.69^{kip} = 680.065^{kip}$$

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 556.875^{kip} + 109.69^{kip} = 666.565^{kip}$$

$$R_n = 680.07^{kip} \not\leq 666.57^{kip} \therefore R_n = 666.57^{kip}$$

$$\phi R_n = 0.75\left(666.57^{kip}\right) = 499.92^{kip}$$

**Tension Member Block Shear Mode 2:**

$$R_n = 0.6F_yA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

Gross area in tension, $A_{gt}$

$$A_{gt} = (6.5'^{in})(0.5625'^{in}\text{ thick}) = 3.656^{in^2}$$

Net area subject to tension, $A_{nt}$

$$A_{nt} = A_{gt} - A_{holes}$$

$$A_{nt} = 3.656^{in^2} - \left(1 + \frac{1}{2}\right)\left(\frac{2}{8} + \frac{1}{10} + \frac{1}{10}\right)(0.5625'^{in}) = 2.813^{in^2}$$

Gross area in shear, $A_{gv}$

$$A_{gv} = 1(16.5'^{in})(0.5625'^{in}) = 9.281^{in^2}$$
Net area subject to shear, $A_{nv}$

$A_{nv} = A_{gv} - A_{holes}$

$A_{nv} = 9.281\text{in}^2 - (1+1+\frac{1}{2})(\frac{7}{8} + \frac{1}{16} + \frac{1}{16}) (0.5625\text{in}) = 7.313\text{in}^2$

$R_u = 0.6F_yA_{nv} + U_{bs}F_yA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_yA_{nt}$

$U_{bs} = 1.0$ as per Commentary to Ch. J, pg. 16.1-352

$0.6F_yA_{nv} = 0.6(65\text{kips})\left(7.313\text{in}^2\right) = 285.19\text{kips}$

$U_{bs}F_yA_{nt} = 1.0(65\text{kips})\left(2.813\text{in}^2\right) = 182.85\text{kips}$

$0.6F_yA_{gv} = 0.6(50\text{kips})\left(9.281\text{in}^2\right) = 278.43\text{kips}$

$0.6F_yA_{nv} + U_{bs}F_yA_{nt} = 285.19\text{kips} + 182.85\text{kips} = 468.04\text{kips}$

$0.6F_yA_{gv} + U_{bs}F_yA_{nt} = 278.43\text{kips} + 182.85\text{kips} = 461.28\text{kips}$

$R_u = 468.04\text{kips} \not\leq 461.28\text{kips} \therefore R_u = 461.28\text{kips}$

$\phi R_u = 0.75\left(461.28\text{kips}\right) = 345.96\text{kips}$

Block Shear Failure of the Gusset Plate:

Gross area in tension, $A_{gt}$

$A_{gt} = (4\text{in})(2\text{pl}) (0.50\text{in thick}) = 4.00\text{in}^2$

Net area subject to tension, $A_{nt}$

$A_{nt} = A_{gt} - A_{holes}$

$A_{nt} = 4.00\text{in}^2 - \left(\frac{1}{2} + \frac{1}{2}\right)(\frac{7}{8} + \frac{1}{16} + \frac{1}{16})(0.50\text{in})(2\text{pl})(0.50\text{in}) = 3.00\text{in}^2$

Gross area in shear, $A_{gv}$

$A_{gv} = 2(15.5\text{in})(2\text{pl})(0.50\text{in}) = 31.00\text{in}^2$
Net area subject to shear, $A_{nv}$

$$A_{nv} = A_{gv} - A_{holes}$$

$$A_{nv} = 31.00\text{in}^2 - (2)(1+1+\frac{1}{2})(\frac{2}{17} + \frac{3}{20} + \frac{3}{16})(2\pi l)(0.50\text{in}) = 24.00\text{in}^2$$

$$R_u = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$U_{bs} = 1.0$ as per Commentary to Ch. J, pg. 16.1-352

$$0.6F_u A_{nv} = 0.6\left(58\text{ksi}\right)\left(24.00\text{in}^2\right) = 835.2\text{kips}$$

$$U_{bs} F_u A_{nt} = 1.0\left(58\text{ksi}\right)\left(3.00\text{in}^2\right) = 174.00\text{kips}$$

$$0.6F_y A_{gv} = 0.6\left(36\text{ksi}\right)\left(31.00\text{in}^2\right) = 669.60\text{kips}$$

$$0.6F_u A_{nv} + U_{bs} F_u A_{nt} = 835.2\text{kips} + 174.00\text{kips} = 1009.2\text{kips}$$

$$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 669.60\text{kips} + 174.00\text{kips} = 843.6\text{kips}$$

$$R_u = 1009.2\text{kips} \not\leq 843.6\text{kips} \therefore R_u = 843.6\text{kips}$$

$$\phi R_u = 0.75\left(843.6\text{kips}\right) = 632.7\text{kips}$$

Serviceability:

$$\frac{L}{r_{min}} \leq 300$$

$$(20')\left(12''\right) \leq 300$$

$$r_{min}$$

$$I_x = \frac{bh^3}{12} = \frac{\frac{9}{10}''\left(9''\right)^3}{12} = 34.2\text{in}^4$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{34.2}{5.0625}} = 2.60$$

$$I_y = \frac{bh^3}{12} = \frac{9''\left(\frac{9}{10}''\right)^3}{12} = 0.133\text{in}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{0.133}{5.0625}} = 0.162$$

$$(20')\left(12''\right)\leq 300$$

$$\frac{0.162''}{0.162''} \leq 300$$

$$1481.5 \geq 300 \therefore$$ This cross-section fails the serviceability check.

Conclusion: NSF of the tension member controls, and $\phi R_u = (0.75)\left(255.97\text{kips}\right) = 191.98\text{kips}$. The section does not meet serviceability requirements.
2. Given the following A992 steel frame, determine the $k$ value that should be used for column EF. Note that joint G is fixed rotationally and translationally, while joint H is fixed rotationally only. All bending may be assumed to occur about the strong axis.

Column EF is laterally unbraced, because joint F is able to translate laterally with respect to joint E. Therefore, use the alignment chart for unbraced frames.

$$G_F = \frac{\sum (EI/L)_{column}}{\sum (EI/L)_{girder}} = \frac{\left(\frac{455}{30}\right)}{(\frac{3000}{40})} = 0.3033$$

$$G_E = \frac{\sum (EI/L)_{column}}{\sum (EI/L)_{girder}} = \frac{\left(\frac{455}{30}\right) + \left(\frac{455}{30}\right)}{(2)(\frac{3000}{40})} = 0.2022$$

$k = 1.16$ (See alignment chart next page)
Fig. C-C2.4. Alignment chart—sideways uninhibited (moment frame).
3. Select the lightest W14 section made of A992 steel to support the following compressive service loads:

Dead Load = 100 kips  
Live Load = 175 kips  
Snow Load = 20 kips  
Roof Live Load = 50 kips  

The column should have a length of 20 ft, and you may assume that $k = 1.0$ for both x- and y-axes.

You may use the design tables included within the manual if appropriate for column selection, but you MUST still check your selection by hand with all appropriate checks. Be sure to provide enough detail in your selection and checking process so that I can assign partial credit if warranted.

**Load Combinations:**

1. $R_u = \sum \gamma_i Q_i = 1.4(D) = 1.4(100) = 140\text{kips}$
2. $R_u = \sum \gamma_i Q_i = 1.2(D) + 1.6(L) + 0.5(L_e \ or \ S \ or \ R) = 1.2(100) + 1.6(175) + 0.5(50) = 425\text{kips}$
3. $R_u = \sum \gamma_i Q_i = 1.2D + 1.6(L_e \ or \ S \ or \ R) + (0.5L \ or \ 0.8W) = 1.2(100) + 1.6(50) + 0.5(175) = 287.5\text{kips}$
4. $R_u = \sum \gamma_i Q_i = 1.2D + 1.6W + L + 0.5(L_e \ or \ S \ or \ R) = 1.2(100) + (175) + 0.5(50) = 320\text{kips}$
5. $R_u = \sum \gamma_i Q_i = 1.2D + 1.0E + L + 0.2S = 1.2(100) + (175) + 0.2(20) = 299\text{kips}$
6. $R_u = \sum \gamma_i Q_i = 0.9D + 1.6W + 1.6H$
7. $R_u = \sum \gamma_i Q_i = 0.9D + 1.0E + 1.6H$

**Column Selection (W14s):**

Entering the W14 selection tables with $(KL)_y = 20'$ and $R_u = 425\text{kips}$, the first section that works is a **W14x68** with a design resistance of 448\text{kips}.

$r_x/r_y = 2.44$

$\frac{(KL)}{r_{eq}} = \frac{20.0^a}{2.44} = 8.20^a$, therefore, y-axis buckling controls.
Check the Capacity of the W14x68:

**Flexural Buckling Capacity:**

\[ P_n = F_{cr} A_y \quad \text{(E3-1)} \]

if \( \left( \frac{KL}{r} \right)_x \leq 4.71 \left( \sqrt[3]{\frac{E}{F_y}} \right) \rightarrow F_{cr} = \left[ 0.658 \left( \frac{F_y}{E} \right) \right] F_y \quad \text{(E3 - 2)} \)

if \( \left( \frac{KL}{r} \right)_y > 4.71 \left( \sqrt[3]{\frac{E}{F_y}} \right) \rightarrow F_{cr} = 0.877 F_c \quad \text{(E3 - 3)} \)

Solve for \( \left( \frac{KL}{r} \right)_{\max} \) – the greatest slenderness ratio - which corresponds to \( r_{\min} \) and \( KL_{\max} \).

\[
\left( \frac{KL}{r} \right)_x = \left( \frac{(20')(12')}{6.01''} \right) = 39.93
\]

\[
\left( \frac{KL}{r} \right)_y = \left( \frac{(20')(12')}{2.46''} \right) = 97.56 \quad \text{Greater slenderness ratio governs}
\]

Compare maximum slenderness ratio against \( 4.71 \left( \sqrt[3]{\frac{E}{F_y}} \right) \) to determine how the column will buckle.

\[
4.71 \left( \sqrt[3]{\frac{E}{F_y}} \right) = 4.71 \left( \sqrt[3]{\frac{29,000}{50}} \right) = 113.43
\]

Since \( \left[ \left( \frac{KL}{r} \right)_x = 97.56 \right] \leq 4.71 \left( \sqrt[3]{\frac{E}{F_y}} = 113.43 \right) \rightarrow F_{cr} = \left[ 0.658 \left( \frac{F_y}{E} \right) \right] F_y \)

Inelastic buckling governs.

\[
P_n = F_{cr} A_y
\]

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

where

\[
F_e = \pi^2 \frac{E}{\left( \frac{KL}{r} \right)} = \pi^2 \frac{(29,000)}{(97.56)^2} = 30.07^{\text{ksi}}
\]

\[
F_{cr} = \left[ 0.658 \left( \frac{50}{30.07} \right) \right] (50^{\text{ksi}}) = 24.93^{\text{ksi}}
\]

\[
P_n = \left( 24.93^{\text{ksi}} \right) (20.0^{\text{in}^2}) = 498.61^{\text{kips}}
\]
Local Buckling:

Flanges:

\[
\lambda_{f,1} = \frac{b_r}{2t_f} = 6.97
\]
\[
\lambda_{r,fl} = 0.56 \sqrt{\frac{E}{F_y}} = 0.56 \sqrt{\frac{29,000}{50}} = 13.49
\]

Since \( \lambda_{f,1} < \lambda_r \), local buckling will not occur in the flanges.

Web:

\[
\lambda_{web} = \frac{h}{t_w} = 27.5
\]
\[
\lambda_{r,web} = 1.49 \sqrt{\frac{E}{F_y}} = 1.49 \sqrt{\frac{29,000}{50}} = 35.88
\]

Since \( \lambda_{web} < \lambda_r \), local buckling will not occur in the web.

Conclusion: Use a W14x68, with \( \phi P_n = 0.90 \left( 498.61^{kips} \right) = 448.75^{kips} \).
Bonus (2 pts): Why is the current edition of the AISC Steel Construction Manual the 13th edition, knowing that a 12th edition of the manual does not exist?