You MUST use the following solution method for credit on these problems.
1) Tell me what you are trying to do – i.e. lay out a new section of your exam for everything you need to do to solve the problem and give that section a title. For example: “Computing $\beta$ for use in the above equation:” “Finding effective Lfb about the weak axis:” Then compute it.
2) Tell me what equation you are using, and which page you got it from.
3) Tell me what table you are utilizing and what page you got it from.
4) As soon as you finish a section of your solution, start a new section and tell me what you are now trying to accomplish.

Without this information, I can assure you that I will not have time to try and dig out what you are attempting to do, and your grade will reflect that. Also, if I cannot read it I will not grade it. Only professional engineering work will be accepted.

Problem 1) Determine the strength of the slip critical connection shown, including all limit states. The bolts are 1 inch diameter A325 with threads excluded from the plane of shear. The channel is a C10x25 of A992 steel and the plate is A992 steel 12"x3/4". S1 = 3", S2 = 2". The connection is a Class B surface. If you feel any information required to solve this problem is missing, simply make a reasonable assumption and state that assumption.

Problem 2) Two plates are welded, as shown. The lower plate is an 8"x1/2" plate of A514 steel, whereas the plate on top is a 6"x1/2" of A514 steel. See page 2-25 for specs on this material. An appropriate weld material is used in the connection. The two plates overlap 10", and no end welds are used. Design the welds to develop the full strength of the connection.

Problem 3) The loads shown on the beam column are factored. $P_1 = 100k, P_2 = 12k$. The beam-column is a W8x40 of A992 steel. $L = 8$ feet. Is this member satisfactory?
Problem 1

Given:
- \( C_{10}x_{25} - A = 7.34 \text{ in}^2 \)
- \( t_w = 0.526 \text{ in} \)
- \( f = 0.617 \text{ in} \)
- \( 50/65 \text{ ksi} \)

G54
\[ \phi P_m = \phi F_y A_g = (50 \text{ ksi})(7.34 \text{ in}^2)(0.9) = 330.3 \text{ k} \]

NSF
\[ A_n = 7.34 - 3(0.526 \text{ in})(1 + \frac{1}{8}) = 5.565 \text{ in}^2 \]
\[ U = 1 - \frac{f}{2} = 1 - \frac{0.617 \text{ in}}{6} = 0.897 \]

\[ A_e = 5.565 \times 0.897 = 4.993 \text{ in}^2 \]

\[ \phi P_m = 0.75 \text{ Fu } A_e = 0.75(65 \text{ ksi})(4.993 \text{ in}^2) = 243.4 \text{ k} \]

B.S.F.: Channel is thinner of same steel controls

Diagram:
- 8" - 6"
- Middle section with holes

\[ A_{gw} = (8") (0.526 \text{ in})(2) = 8.416 \text{ in}^2 \]
\[ A_g t = (6") (0.526 \text{ in}) = 3.156 \text{ in}^2 \]
\[ A_{nw} = 8.416 - (7 \text{ holes })(1\frac{1}{8}) (0.526 \text{ in}^2) = 4.273 \text{ in}^2 \]
\[ A_{nt} = 3.156 - (1 \text{ hole })(1\frac{1}{8}) (0.526 \text{ in}^2) = 1.9725 \text{ in}^2 \]
Block Shear Rupture: DO NOT LET LOWERY CATCH YOU WITH THIS EXAM!

\[ A_{gy} = 2 \left( 8'' \right) (0.526'') = 8.42 \text{ in}^2 \]
\[ A_{gt} = 6'' (0.526'') = 3.16 \text{ in}^2 \]
\[ A_{nv} = 8.42 \text{ in}^2 - 2 \text{ holes} (1/8'')(0.526'') = 4.28 \text{ in}^2 \]
\[ A_{nt} = 3.16 \text{ in}^2 - 2 \text{ holes} (1/8'')(0.526'') = 1.98 \text{ in}^2 \]

\[ F_u A_{nt} = 65 \text{ ksi} \left( 1.98 \text{ in}^2 \right) = 128.7 \text{ kips} \]

\[ < 0.6 F_u A_{nv} = 0.6 \left( 65 \text{ ksi} \right) (4.28 \text{ in}^2) = 166.92 \text{ kips} \]

Use Eq 5.4.36

\[ \varphi P_n = \varphi \left[ 0.6 F_u A_{nv} + F_y A_{gt} \right] = \varphi \left[ 0.6 F_u A_{nv} + F_u A_{nt} \right] \]

\[ \varphi = 0.75 \]

\[ 0.75 \left[ 166.92 \text{ kips} + 50 \text{ ksi} (3.16 \text{ in}^2) \right] = 243.69 \text{ kips} \]

\[ \leq 0.75 \left[ 166.92 \text{ kips} + 65 \text{ ksi} (1.98 \text{ in}^2) \right] = 221.72 \text{ kips} \]

\[ \varphi P_n = \boxed{221.72 \text{ kips}} \]

Connection strength limited by Block Shear to 221.72 kips

\[ \checkmark \]
Shear strength one bolt:
\[ \phi R_b = \pi (1)^2/4 = 0.785 \text{in}^2 \]
\[ \phi R_n = 0.75(60 \text{ksi})(0.785 \text{in}^2) = 35.34^k \]
For 12 bolts:
\[ \phi R_n = 424^k \]

Slip critical strength
\[ \phi \mu = 1.0(1.13)(0.5)(512)(N_b = 1)(12 \text{bolts}) = 345,78^k \]

Bearing strength
Channelweb is thinner so critical

\[ h = 1" + \frac{1}{16} = 1.0625 \text{in} \]

for holes nearest end
\[ L_c = L_e - \frac{h}{2} = 2" - \frac{1.0625}{2} = 1.469" \]
\[ \phi R_n = 0.75(1.2)(1.469^\prime)(0.526 \text{in})(65 \text{ksi}) = 45.20^k/\text{bolt} \]
Upper limit = 0.75(2.4)(1')(0.526in)(65ksi) = 61.54^k/80lt > 45.20 use 45.20

for other holes
\[ L_c = 2 - h = 2 - 1.0625 = 0.9375" \]
\[ \phi R_n = 0.75(1.2)(0.9375')(0.526)(65) = 28.85^k/\text{bolt} \]
Upper limit = 61.54^k see above > 28.85 use 28.85^k/80lt

Bearing strength:
\[ \phi R_n = 3(45.20^k) + 9(28.85^k) = 395.2^k \]
Problem 2

For a 5/16 plate (pg 2-25) $F_y = 100$ ksi

Strength of weaker/smaller plate:

\[ \Phi P_m = \Phi \cdot F_y \cdot A_g = 0.9 \times (100 \text{ ksi}) \times (3 \text{ in}^2) = 270 \text{ k} \]

NSF

\[ 2w = 12" \quad l = 10" \quad w = 0.87 \]

\[ 1.5w = 9" \]

\[ 1.0w = 6" \]

\[ \Phi P_m = 0.75(F_u A_e) = 0.75(110 \text{ ksi})(3w * 0.87) = 215.3 \text{ k} \quad \text{Design for max available load.} \]

Weld length \( L = 20 \text{ in} \)

Weld strength \( = \Phi (0.707W F_w) \)

\[ = 0.75(0.707)(w)(0.6 \times 80 \text{ ksi}) = 25.45 \text{ k/in} \]

So solve for weld size,

\[ 215.3 \text{ k} = 25.45W \frac{k}{\text{in}} \times 20 \text{ in} \]

\[ W = 0.427 \text{ in} \quad \text{use } \frac{7}{16} \text{" weld} \]

Check Base metal:

\[ \Phi R_m = 0.9(0.6 F_y) + L \]

\[ = 0.9(0.6 \times 100 \text{ ksi})(1/2")(20") \]

\[ = 540 \text{ k} > 215.3 \text{ k} \quad \text{OK} \]

Check minimum weld size pg 16.1-54

3/16" minimum 20 OK
Problem 3

\[ M_{nt} = \frac{P L}{4} = \frac{12K(16 \text{ ft})}{4} = 48 \text{ kft} \]

Compute compressive strength

\[ \frac{K L}{A_y} = \frac{1.0(16 \text{ ft})(12)}{2.04} = 94.12 \quad \text{ksi} \]

\[ \phi_c F_a = 22.2 \text{ ksi} \]

\[ P_y = 16,1 \text{ ksi} \]

\[ \phi_c P_m = 0.85 A_y F_a = 22.2 \text{ ksi} (11.7 \text{ in}^2) = 259.9 \text{ k} \]

\[ P_e = \frac{100}{\phi_c P_m} = 0.385 > 0.2 \quad \text{Use AISC Eq H1-1a} \]

Compute bending strength

Is shape compact?

\[ \gamma = 0.38 \sqrt{\frac{29000}{50}} = 9.152 > 7.21 \quad \text{so flange compact} \]

\[ L_6 = 16 \text{ ft} \]

\[ L_p = 1.76 r_v \sqrt{\frac{E}{F_v}} = 7.2 \text{ ft} = 86.47 \text{ in} \]

\[ L_r = 26.446 \text{ ft} = 317.35 \text{ in} \]

\[ M_p = 1990 \text{ k in} = 165.833 \text{ k ft} \]

\[ M_r = 1420 \text{ k in} = 118.333 \text{ k ft} \]

\[ C_b = 1.32 \]

\[ M_n = 2282 > 1990 \quad \Rightarrow \text{ use } 1990 \]

\[ \phi M_n = 149.25 \text{ ft} = 1791 \text{ k in} \]

\[ \frac{100}{259.9} + \frac{0.38}{0.2} \left( \frac{52.65}{149.25} \right) \leq 1 \]

\[ 0.385 + 0.314 = 0.698 \leq 1 \quad \text{OK} \]
Problem 3

AMPLIFICATION

DO NOT LET LOWERY CATCH YOU WITH THIS EXAM!

Given:

\[ k_y = k_x = 1.0 \]

45\% shear All loads are factored.

\[ F_y = 50 \text{ kips} \]

Axial work axis request = 100k = \( P_n \).

Calculate work axis supply.

Will not buckle about strong axis.

\[
\lambda_c = \frac{k_y L}{E} = \frac{(0.458)^2(11.7)^2(26.14 \text{kips})}{1133.57} = 1.24 \leq 1.5
\]

Since \( \lambda_c \leq 1.5 \) use:

\[
F_{cr} = (0.458)^2 F_y = (0.458)^2(50 \text{kips}) = 26.14 \text{kips}
\]

\[
\phi_c P_n = \phi_c A_f F_{cr} = (0.85)(11.7^2)(26.14 \text{kips}) = 260 \text{k}
\]

Calculate moment request:

\[
\text{MOMENT AMPLIFICATION FACTOR } B = \frac{1}{1 - \frac{P_n}{P_c}}
\]

\[ P_c = \text{Euler buckling load.} \]

\[ P_c = \frac{\pi^2 EI_x}{(KL)^2} = \frac{\pi^2 (23000 \text{kips})(146 \text{ in}^3)}{(1.0)(1064 \text{ in}^2 12\%))^2} = 4133.57 \text{k} \]

\[ B = \frac{1}{1 - \frac{P_n}{P_c}} = 1.097 \]

√
DO NOT LET LOWERY CATCH YOU WITH THIS EXAM!

Moment on Beam = \( M_{ax} \)

\[
M_{ax} = \frac{PL}{4} = \frac{(12^4)(16 \times 12)}{4} = 576 \text{ k}\cdot\text{in}
\]

\[ M_u = 8 M_{ax} = \frac{631.2}{3} \text{ k}\cdot\text{in} \text{ - Requested} \]

\[ \text{Strong Axis Supply} : \]

Check Compactness:

\[
\lambda_f = \frac{I_f}{W_{fc}} = 7.21 \text{ - PART A of Manual, (Table 1-1)}
\]

\[
\lambda_p = 0.88 \sqrt{\frac{E}{F_y}} = 0.88 \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 9.15
\]

\[ \lambda_f < \lambda_p \text{ SHAPE IS COMPACT.} \]

Since this Shape is Tabulated \( \Rightarrow \) Use \( \lambda_p \) is compact.

\( \therefore \) Lateral Torsional Buckling will control:

\[
M_n = C_b (M_p - (M_p - M_r) \left( \frac{L_b - L_p}{L_p - L_r} \right))
\]

Calculate \( C_b \).

\[ C_b = 1.32 \] P.S. 15x (Steel) Figure 5.15(c).

Calculate Lengths and Moments:

\[ L_b = 16 \text{ ft} \]

\[
L_p = 1.76 \sqrt{\frac{E}{F_y}} = 1.76 (20.04) = 29000 = 7.206 \text{ ft}
\]

\[
L_r = \frac{F_y X_t}{F_y - F_r} = \sqrt{1 + \frac{X_t}{X_r} (F_y - F_r)^2} = 26.446 \text{ ft}
\]

\[ M_p = F_y S_x = (50 \text{ ksi}) (378 \text{ in}^2) = 1990 \text{ k}\cdot\text{in} \]

\[ M_r = (F_y - F_r) S_x = (40 \text{ ksi}) (355.5 \text{ in}^2) = 1420 \text{ k}\cdot\text{in} \]

\[
M_n = 1.32 \left[ 1990 - (1990 - 1420) \left( \frac{16 - 7.206}{26.446 - 7.206} \right) \right] = 2282 > M_p = 1990 \text{ k}\cdot\text{in}
\]

\[ \therefore M_p = 1990 \text{ k}\cdot\text{in} \]
\[ \phi M_n = 0.90 \ M_p = 1791 \ \text{k-in} \]

Determine which interaction equation to use.

\[ \frac{P_m}{\phi_k P_n} = \frac{100}{240} = 0.417 > 0.2 \]

Use

\[ \frac{P_m}{\phi_k P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_k M_{ux}} + \frac{M_{uy}}{\phi_k M_{uy}} \right) \leq 1.0 \]

Page 231 (sec. 4.4.1)

AISC Equation H1-1a

\[ 0.385 + \frac{8}{9} \left( \frac{631.9}{1791} + 0 \right) = 0.699 < 1.0 \]

This beam is satisfactory.