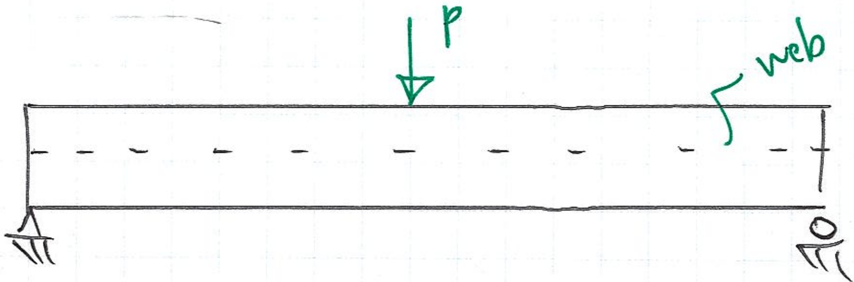
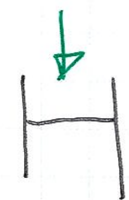
BENDING OF I-SHAPED (& CHANNEL) ABOUT THEIR MINOR AXIS

WEAK, Y-Y





AISC F6, pg 16.1-56

LTB is not a limit state

"compact" section: CASE 13: Table B4.1b

$$\lambda = \frac{b_f}{2t_f} < \lambda_p = 0.38\sqrt{\frac{E}{f_y}} = \frac{65}{\sqrt{f_y}}$$

$$M_n = M_{p,y} = f_y Z_y \leq \underbrace{1.6 f_y S_y}$$

limits excessive deformation which can cause rupture

shape factor

$$\frac{\cancel{f_y} Z_y}{\cancel{f_y} S_y} \geq 1.6$$

* for "almost" all W-shapes $\frac{Z_y}{S_y} \geq 1.6 \therefore \phi_b M_{p,y} = \phi_b M_{n,y}$

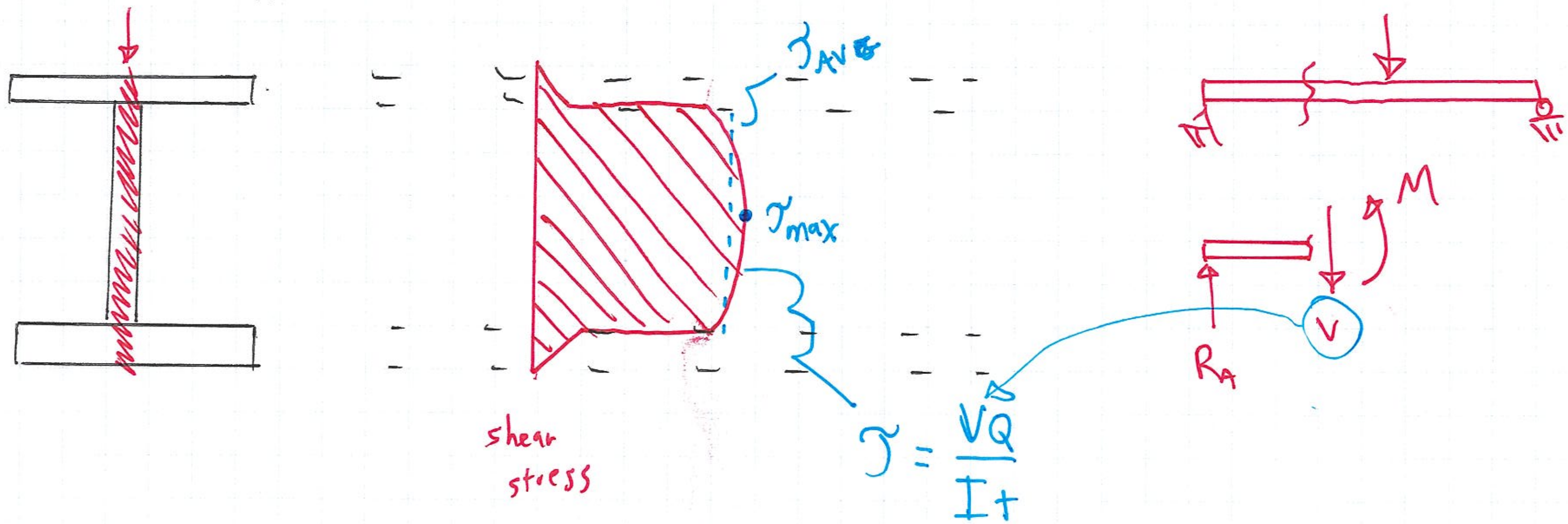
SHEAR CAPACITY OF BEAM

- Shear does not typically control the design (esp. true for rolled shape)
- Design for moment

$$(M_u)_{max} \leq \phi_b M_n$$

- Check for shear

$$(V_u)_{max} \leq \phi_v V_n$$



$$\tau_{AVG} \approx \tau_{MAX} \approx \frac{V}{A_{web}}$$

AISC CHAP G, p16.1 - 70

LIMIT STATE → WEB YIELDING

$$\tau_{yielding} = 0.6f_y$$

$$V_n = \underbrace{0.6f_y}_{\tau_{yield}} A_w C_{v1}$$

C_{v1} ← Web Shear Coef.

A_w: Area of web = $t_w d$

$$V_u = \underbrace{\phi_v V_n}_{\text{design shear strength}}$$

V_u: from V&M diagram (factored shear requirement)

DEFLECTIONS

- Excessive deflections can cause issues w/
  - occupant discomfort
  - ponding water
  - Serviceability
- Deflections are based on service load (not factored)

See AISC pg 3-212 through 227 for defl. formulas

See Segui Table 5.4, p 225 for defl. limits

** TYP. USE LL ONLY

BEAM DESIGN

1) Determine, M_u, for controlling LC for each unbraced length, L_{bi}, of the beam

*** DRAW V&M DIAGRAM

2) Select a trial size:

a) If beam is "fully" braced ($L_b = 0$, $L_b < L_p$) no LTB

$$\left(Z_{req'd}\right) < \frac{(M_u)_{max}}{f_y \phi_b} = \frac{(M_u)_{max}}{0.9 f_y}$$

Table 3-2, p3-19

Beam Tables

$\phi_b M_p > (M_u)_{max}$

*BOLD IS LIGTEST

b) If beam is <u>not</u> fully braced $(L_b > L_p)$ then LTB may control

- $C_{b,i}$ for each unbraced length
- $M_{eq,i} = \dfrac{(M_u)_i}{C_{bi}}$ for each unbraced length
- Select a section from beam <u>charts</u> so that

$$\phi_b M_{ni} \geq M_{eqi} \ @ \ L_{bi}$$

Table 3-10, P3-99
$(f_y = 50 \text{ ksi}, \ C_b = 1.0)$

* SOLID LINES ARE LIGTHEST SECTIONS

3) Verify Cap. of Section

$\phi_b M_p \geq (M_u)_{max}$ "Basic Strength"

$\phi_b M_{ni} \geq (M_{ui})_{max}$ "LTB Strength"

Add dead wt. from beam selected

4) Check deflections

$$\Delta_{max} \le \Delta_{allow}$$ ← Segui Table 5.4

AISC p 3-213 - 227 or derive

5) Check shear

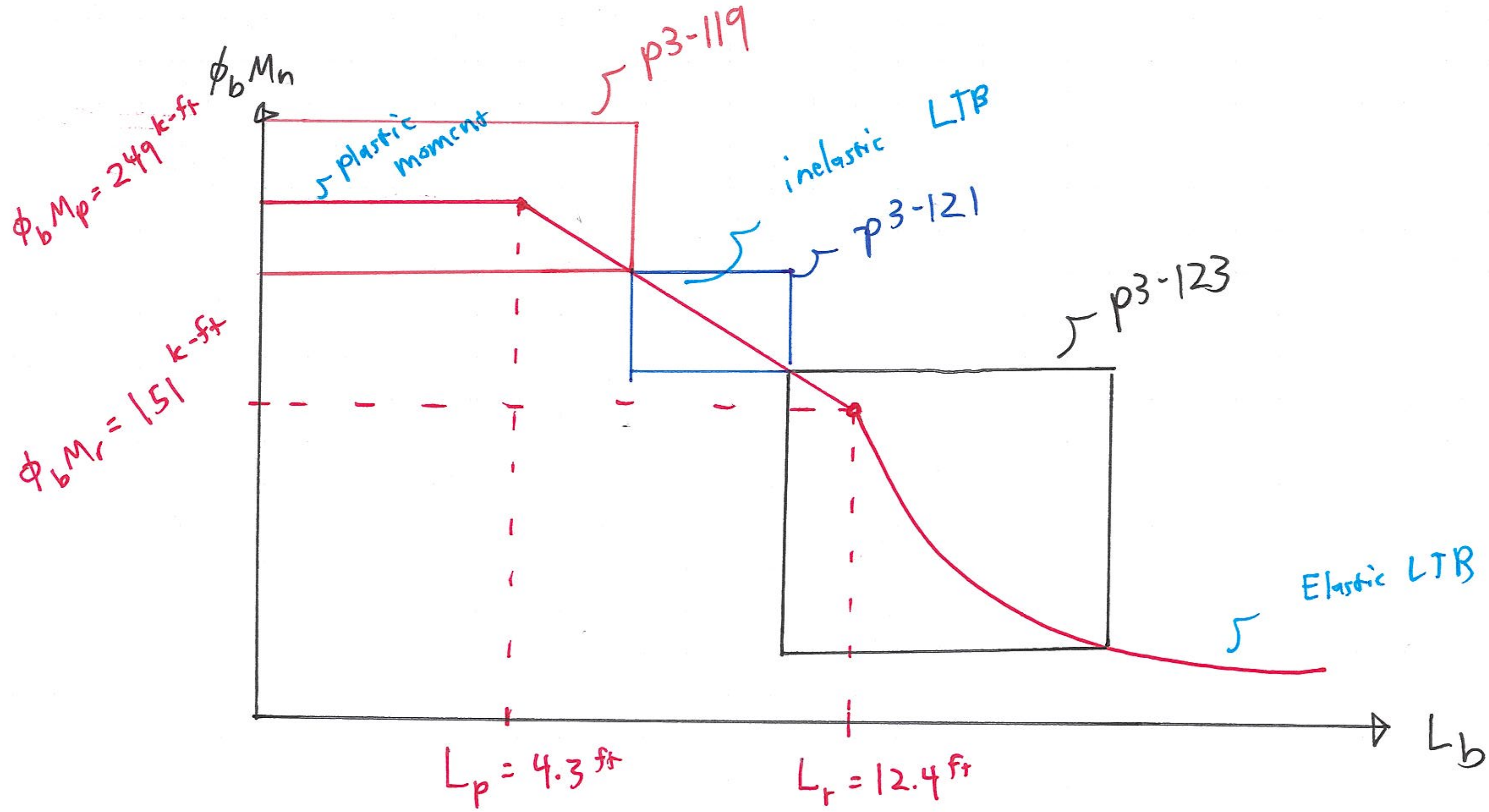
$$(V_u)_{max} \le \phi_v V_n$$

6) Revise Selection if necessary

BEAM DESIGN CHARTS

- small window of the response curve for a given beam (W-shape)

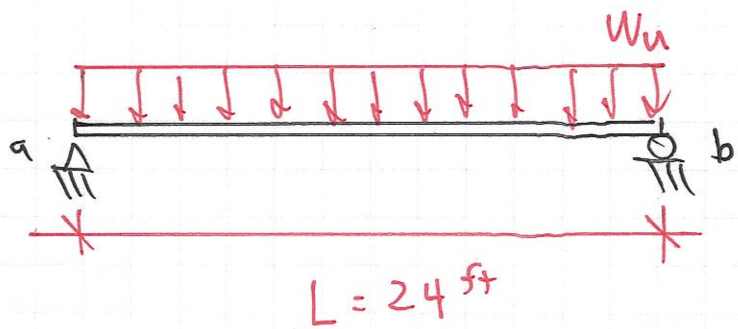
Example W18X35, $f_y = 50$ ksi ; $C_b = 1.0$



EXAMPLE

GIVEN: A continuously braced beam (floor beam) is loaded as shown:

$L_b = 0$, no LTB
plastic moment



$W_L = 4^{k/ft}$

$W_D = 2^{k/ft}$ ← does not include beam wt.

REQ'D: use A992 steel
1) Select the lightest W-Section
2) Repeat if laterally braced at a & b only

$$W_u = 1.2\left(2^{k/ft}\right) + 1.6\left(4^{k/ft}\right) = 8.8^{k/ft}$$

$$M_{max} = \frac{WL^2}{8} = \frac{\left(8.8^{k/ft}\right)\left(24^{ft}\right)^2}{8} = 633.6^{k\text{-}ft}$$

$$\left(Z_{req'd}\right) = \frac{\left(M_u\right)_{max}}{\phi_b F_y} = \frac{\left(633.6^{k\text{-}ft}\right)(12)}{0.9\left(50^{ksi}\right)} \geq 169\ in^3$$

Try W24X68: $Z = 177\ in^3 \geq 169\ in^3$