Problem 1
Consider a built-up flexural section composed of a W27x94 (A992, $F_y = 50$ ksi) and a C15x33.9 (A36, $F_y = 36$ ksi) cap channel. Assume that the components have been welded such that the built-up section will act as a single unit.

Part 1: Calculate the elastic neutral axis location and moment of inertia for strong axis bending. You may ignore fillets. Sketch the cross-section, indicating the elastic neutral axis location. Compare your calculated values (percent difference) to tabulated values in Table 1-19 of the AISC Manual.

Note: We can consider C-shape and W-shape as single elements to calculate ENA

C15X33.9

Depth (Table 1-5) $d_1 := 15\ in$

Thickness (Table 1-5) $t_{w1} := 0.40\ in$

Width of flange (Table 1-5) $b_{f1} := 3.4\ in$

Avg. thickness of flange (Table 1-5) $t_{f1} := 0.65\ in$

Area (Table 1-5) $A_1 := 10\ in^2$

Moment of inertia for y-y (Table 1-5) $I_y := 8.07\ in^4$

x_bar (Table 1-5) $x_{bar} := 0.788\ in$

Note: Considering x_bar as the center of the C-shape welded to W-shape

W-shape W27x94

Depth (Table 1-1) $d_2 := 26.9\ in$

Thickness of web (Table 1-1) $t_{w2} := 0.49\ in$

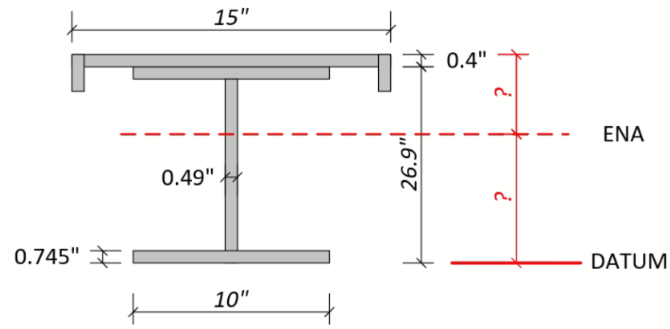
Width of flange (Table 1-1) $b_{f2} := 10\ in$

Thickness of flange (Table 1-1) $t_{f2} := 0.745\ in$

Area (Table 1-1) $A_2 := 27.6\ in^2$

Moment of inertia for x-x (Table 1-1) $I_x := 3270\ in^4$

Determine the ENA for this shape



$$Y_{ENA} = \frac{\sum Ay}{\sum A}$$

Total H for the built-up section $H := d_2 + t_{w1} = 27.3\ in$

y for the C-shape to the DATUM $y_1 := H - x_{bar} = 26.512\ in$

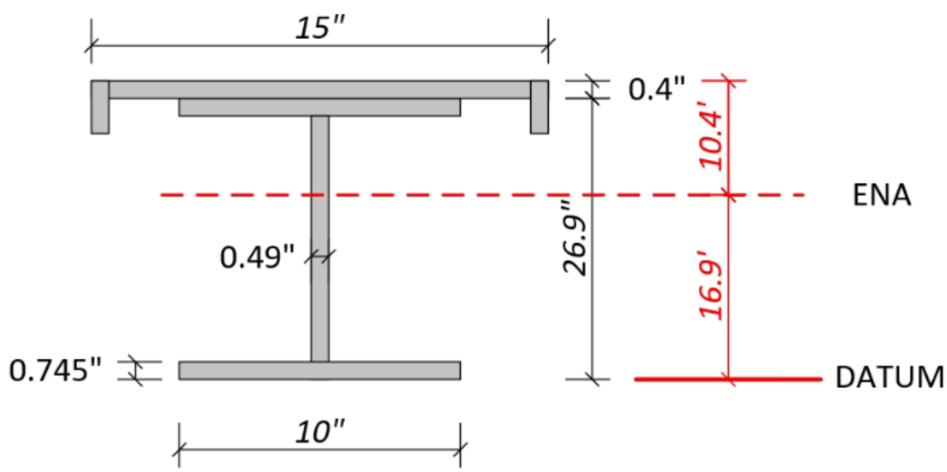
y for the W-Shape to the DATUM $y_2 := \frac{d_2}{2} = 13.45\ in$

Summation of A $A := A_1 + A_2 = 37.6\ in^2$

Summation of Ay $Ay := A_1 \cdot y_1 + A_2 \cdot y_2 = 636.34\ in^3$

Y_ENA to the DATUM $Y_{ENA} := \frac{Ay}{A} = 16.9\ in$

Y_ENA to the top of section $H - Y_{ENA} = 10.4\ in$



Determine the moment of inertia for strong axis bending for this shape

Distance for the center of C-shape to the Y_ENA $D_1 := y_1 - Y_{ENA} = 9.588\ in$

Distance for the center of W-shape to the Y_ENA $D_2 := y_2 - Y_{ENA} = -3.474\ in$

Io+AD^2 (in^4) for the C-shape $I_1 := I_y + A_1 \cdot {D_1}^2 = 927.38\ in^4$

Io+AD^2 (in^4) for the W-shape $I_2 := I_x + A_2 \cdot {D_2}^2 = 3603.083\ in^4$

Moment of inertia about strong axis $I := I_1 + I_2 = 4530.463\ in^4$

Compare calculated values (percent difference) to tabulated values in Table 1-19

Y_ENA to the DATUM (Table 1-19) $Y_{1\_table} := 16.9\ in$

Y_ENA to the top of section (Table 1-19) $Y_{2\_table} := 10.4\ in$

Moment of inertia about strong axis (Table 1-19) $I_{table} := 4530\ in^4$

Percentage differences

$$\frac{Y_{ENA}}{Y_{1\_table}} = 100.142\%$$

$$\frac{H - Y_{ENA}}{Y_{2\_table}} = 99.77\%$$

$$\frac{I}{I_{table}} = 100.01\%$$

Part 2: What is the nominal yield moment of the built-up section, neglecting residual stresses (i.e., assume $F_r = 0$ ksi)?

yield stress in C-shape (top) $F_{y1} := 36\ ksi$

yield stress in W-shape (bottom) $F_{y2} := 50\ ksi$

Section of modulus (top) $S_{xt} := \dfrac{I}{H - Y_{ENA}} = 436.6\ in^3$

Section of modulus (bottom) $S_{xb} := \dfrac{I}{Y_{ENA}} = 267.7\ in^3$

The nominal yield moment

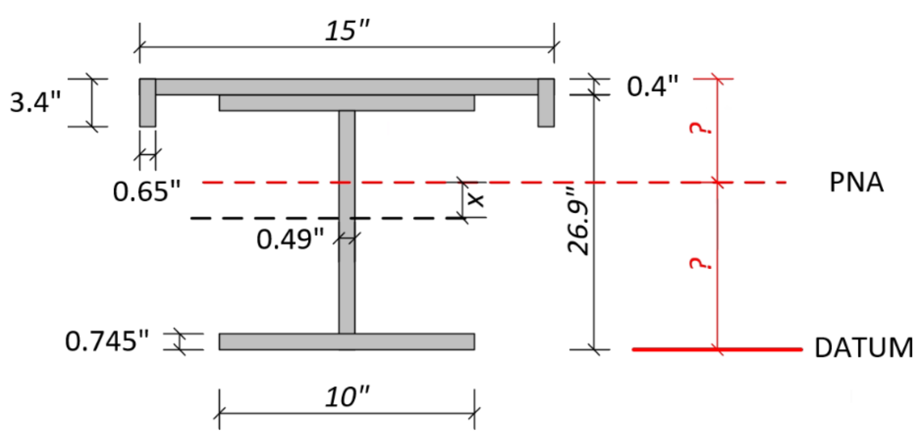
The nominal yield moment for the top $M_{yt} := F_{y1} \cdot S_{xt} = 1309.9\ kip \cdot ft$

The nominal yield moment for the bottom $M_{yb} := F_{y2} \cdot S_{xb} = 1115.4\ kip \cdot ft$

The nominal yield moment $M_y := M_{yb} = 1115.4\ kip \cdot ft$

Note: Therefore, the bottom yield moment governs (reaches yield first).

Part 3: Calculate the plastic neutral axis location and plastic moment. Sketch the cross-section, indicating the plastic neutral axis location. What is the *effective shape factor*, ξ_{Xeff}, of the built-up section in strong axis flexure (i.e., what is the ratio of the plastic moment to the yield moment)? .

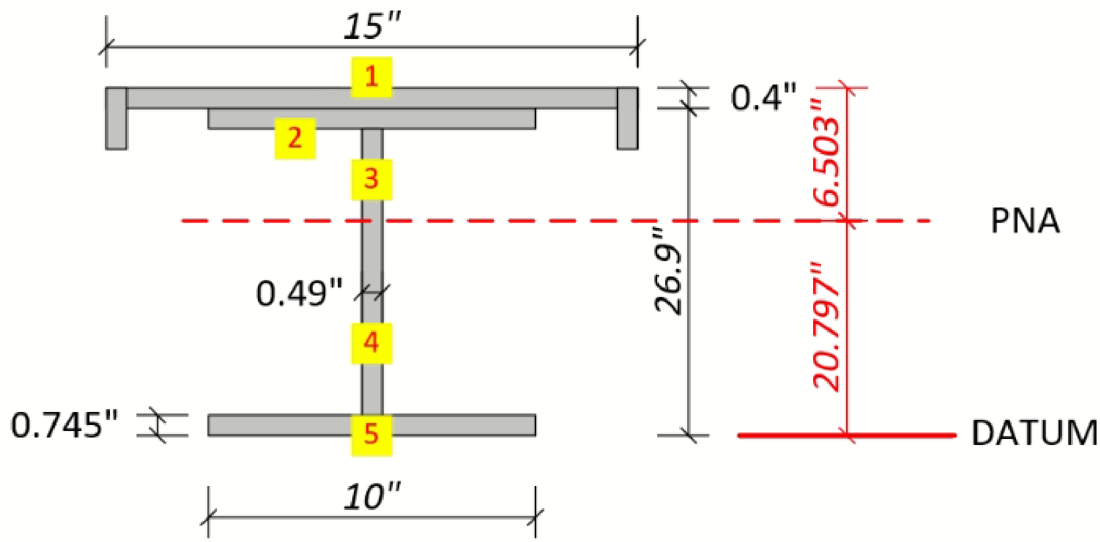


Assume PNA at the middle of the W-shape considering no C-shape welded on it. The C-shape welded on the top of the W-shape, therefore, the PNA would be higher than the middle of the W-shape. Assume PNA is in the W-shape web. Assume the "X" distance between PNA to the middle of W-Shape.

Determine the distance X $X := \frac{\left(F_{y1} \cdot A_1\right)}{2 \cdot F_{y2} \cdot t_{w2}} = 7.347 \ in$

Y_PNA to the DATUM $Y_{PNA} := \frac{d_2}{2} + X = 20.797 \ in$

Y_PNA to the top of section $H - Y_{PNA} = 6.503 \ in$



Assume the top is in compression (Shape 1, 2 & 3)

yield stress in C-shape $F_{y1} := 36 \ ksi$

yield stress in W-shape $F_{y2} := 50 \ ksi$

Area of the shape 1 (C-shape) $A_{s1} := A_1 = 10 \ in^2$

Area of the shape 2 (W-shape flange) $A_{s2} := b_{f2} \cdot t_{f2} = 7.45 \ in^2$

Area of the shape 3 (part of web) $A_{s3} := \left(d_2 - t_{f2} - Y_{PNA}\right) \cdot t_{w2} = 2.625 \ in^2$

y for the center of shape 1 to the DATUM $y_{s1} := H - x_{bar} = 26.512 \ in$

y for the center of shape 2 to the DATUM $y_{s2} := d_2 - t_{f2} \cdot 0.5 = 26.528 \ in$

y for the center of shape 3 to the DATUM $y_{s3} := \left(d_2 - t_{f2} - Y_{PNA}\right) \cdot 0.5 + Y_{PNA} = 23.476\ in$

Distance of the center of shape 1 to Y_PNA $D_{s1} := \text{abs}\left(y_{s1} - Y_{PNA}\right) = 5.715\ in$

Distance of the center of shape 2 to Y_PNA $D_{s2} := \text{abs}\left(y_{s2} - Y_{PNA}\right) = 5.731\ in$

Distance of the center of shape 3 to Y_PNA $D_{s3} := \text{abs}\left(y_{s3} - Y_{PNA}\right) = 2.679\ in$

Assume the bottom is in tension (Shape 4 & 5)

Area of the shape 4 (part of web) $A_{s4} := t_{w2} \cdot \left(d_2 - 2 \cdot t_{f2}\right) - A_{s3} = 9.825\ in^2$

Area of the shape 5 (W-shape flange) $A_{s5} := b_{f2} \cdot t_{f2} = 7.45\ in^2$

y for the center of shape 4 to the DATUM $y_{s4} := \left(Y_{PNA} - t_{f2}\right) \cdot 0.5 + t_{f2} = 10.771\ in$

y for the center of shape 5 to the DATUM $y_{s5} := t_{f2} \cdot 0.5 = 0.373\ in$

Distance of the center of shape 4 to Y_PNA $D_{s4} := \text{abs}\left(y_{s4} - Y_{PNA}\right) = 10.026\ in$

Distance of the center of shape 5 to Y_PNA $D_{s5} := \text{abs}\left(y_{s5} - Y_{PNA}\right) = 20.424\ in$

The nominal plastic moment, **forces times arms ($M_p = F_y * A_i * D_i$)**

$$M_p := F_{y1} \cdot A_{s1} \cdot D_{s1} + F_{y2} \cdot A_{s2} \cdot D_{s2} + F_{y2} \cdot A_{s3} \cdot D_{s3} + F_{y2} \cdot A_{s4} \cdot D_{s4} + F_{y2} \cdot A_{s5} \cdot D_{s5} = 1423.1\ kip \cdot ft$$

Shape factor $\xi_{X\_eff} := \dfrac{M_p}{M_y} = 1.276$

Part 4: What is the lightest rolled A992 W-shape that would provide the same plastic moment capacity as this built-up section? (Hint: Use Table 3-2. Note that you calculated in Part 3 is the nominal *Mp*)

Phi factor $\phi_b := 0.9$

Factored moment capacity $\phi_b M_p := \phi_b \cdot M_p = 1280.8\ kip \cdot ft$

Note: Then, use Table 3-2 to find the lightest rolled A992 W-shape is **W30X108** (bolded value on Pg. 3-23). **W30X108 has** $\phi_b M_p$ = 1300 kip*ft.

Part 5: What is the weight savings of the lightest rolled section relative to the built-up section? Why might the built-up section be preferable to the lightest rolled section, even though it is more expensive in terms of both material and fabrication?

Weight of built-up section (Table 1-19) $W_1 := 0.128\ \boldsymbol{klf}$

Weight of W-shape section (Table 1-1) $W_2 := 0.108\ \boldsymbol{klf}$

Save weight percentage by using W-shape $1 - \frac{W_2}{W_1} = 15.625\%$

Main reason to use this built-up section

1. The allowable unbraced length increases (rt increases, Lp, Lr increases, strength figure of LTB shift to the right) which is good for LTB.

Problem 2

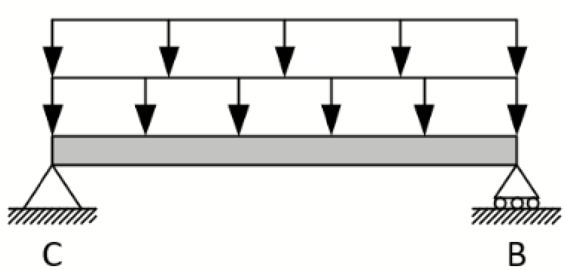
Refer to the structure and loading from Assignment 1. Do not consider live load reduction. Do not specify camber, but ensure that your designs satisfy serviceability limits of L/240 for total load deflection and L/360 for live load deflection. Girder deflections may be obtained from software. If girder deflections are calculated by hand, it is acceptable to use either a distributed load or concentrated loads. Design W-shapes of the preferred material specification for:

Part 1: A simple-span typical beam at the 3rd floor, in the bay bounded by column lines B to C and 30 to 32. Note that typical floor beams are continuously laterally supported by the floor deck.

Recommend Design Steps for continuously laterally beam (L_b = 0) case

1. Compute factored moment and shear, M_u & V_u.Assume a beam self-weight (i.e., 50 or 100 lb/ft)
2. Compute required I_x based on deflection limits. Be careful with units.
3. Use Table 3-2, select the lightest section based on $\phi_b M_{px}$ values if F_y = 50 ksi.
4. If I_x for the trial section is smaller than the required I_x, refer to Table 3-3 and select the most economical section based on I_x. (Skip if $I_{x\_trial} \geq I_{x\_req}$)
5. Analyze the trial section to verify capacity
    - Moment capacity (if F_y= 50 ksi, you can use the Table 3-2 and verify $\phi_b M_{px}$. If the beam is heavier than what you assumed, you may need to recalculate M_u and verify again)
    - Shear capacity (if F_y= 50 ksi, you can use the Table 3-2 and verify $\phi_v V_n$)
    - Deflection limits (checking deflection is not necessary if you do not have calculation errors before)

Determine the factored moment and shear, M_u & V_u (Refer to Project 1 B solution)



Floor beam length $L := 30\ ft$

Tributary width $W := 10\ ft$

Dead load on the floor beam $w_D := 75\ psf \cdot W = 0.75\ klf$

Live load on the floor beam $w_L := 100\ psf \cdot W = 1\ klf$

Assume 50 lb/ft self-weight of the floor beam $w_O := 50\ plf = 0.05\ klf$

Mid-span dead load moment $M_{Mid\_D} := (w_D + w_O) \cdot \frac{L^2}{8} = 90\ kip \cdot ft$

End support dead load shear $\qquad V_{End\_D} := (w_D + w_O) \cdot \dfrac{L}{2} = 12 \ \boldsymbol{kip}$

Mid-span live load moment $\qquad M_{Mid\_L} := w_L \cdot \dfrac{L^2}{8} = 112.5 \ \boldsymbol{kip \cdot ft}$

End support live load shear $\qquad V_{End\_L} := w_L \cdot \dfrac{L}{2} = 15 \ \boldsymbol{kip}$

Control load combination **1.2D + 1.6L** *(Project 1B solution)*

Factored moment at the mid $\qquad M_u := 1.2 \cdot M_{Mid\_D} + 1.6 \cdot M_{Mid\_L} = 288 \ \boldsymbol{kip \cdot ft}$

Factored shear at the end $\qquad V_u := 1.2 \cdot V_{End\_D} + 1.6 \cdot V_{End\_L} = 38.4 \ \boldsymbol{kip}$

Determine the required moment of inertia, I_x

Modulus of elasticity of steel $\qquad E := 29000 \ \boldsymbol{ksi}$

Max allowable deflection at the mid for live load $\qquad \Delta_{L\_max} := \dfrac{L}{360} = 1 \ \boldsymbol{in}$

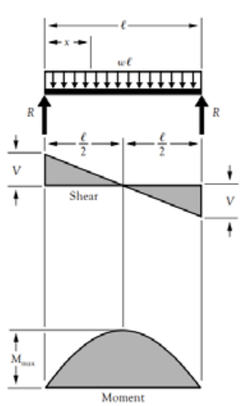
Max allowable deflection at the mid for total load $\qquad \Delta_{T\_max} := \dfrac{L}{240} = 1.5 \ \boldsymbol{in}$

Required moment of inertia of live load $\qquad I_{x\_L} := \dfrac{(5 \cdot w_L \cdot L^4)}{384 \cdot E \cdot \Delta_{L\_max}} = 628.448 \ \boldsymbol{in}^4$

Required moment of inertia of total load $\qquad I_{x\_T} := \dfrac{(5 \cdot (w_D + w_O) \cdot L^4 + 5 \cdot w_L \cdot L^4)}{384 \cdot E \cdot (\Delta_{T\_max})} = 754.138 \ \boldsymbol{in}^4$

Required moment of inertia (maximum) $\qquad I_{x\_req} := \max(I_{x\_L}, I_{x\_T}) = 754.138 \ \boldsymbol{in}^4$

Figure 1 Simple Beam – Uniformly Distributed Load



$R = V \ldots\ldots = \dfrac{w\ell}{2}$

$V_x \ldots\ldots = w\left(\dfrac{\ell}{2} - x\right)$

M_{max} (at center) $\ldots\ldots = \dfrac{w\ell^2}{8}$

$M_x \ldots\ldots = \dfrac{wx}{2}(\ell - x)$

Δ_{max} (at center) $\ldots\ldots = \dfrac{5w\ell^4}{384\,EI}$

$\Delta_x \ldots\ldots = \dfrac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$

Use Table 3-2, select the lightest section based on $\phi b M_{px}$ values and assume F_y = 50 ksi

Find the lightest section with $\phi_b M_{px} >= M_u$ (**Table 3-2)**

Try **<u>W18X40</u>** (bolded on page 3-26) with $\phi_b M_{px}$ = 294 kip-ft. **Bold** means it is the lightest in that group.

Check if selected trial section has $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 612\ in^4$

$$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{"OK"}, \text{"NG"}\right) = \text{"NG"}$$

Then, refer to Table 3-3 and select the most economical section based on I_x.Therefore, try **<u>W21X44</u>** (bolded on page 3-29).

Analyze the trial section to verify capacity

Check if $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 843\ in^4$

$$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Check if $\phi_b M_{px} \geq M_u$

Yield stress (Table 1-12) $F_y := 50\ ksi$

Modulus of elasticity E $E := 29000\ ksi$

Plastic section modulus (Table 1-1) $Z_x := 95.4\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 7.22$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\dfrac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\dfrac{E}{F_y}} = 24.083$

$$check := \text{if}\left(\lambda_f \leq \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 53.6$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$$check := \textbf{if}\left(\lambda_w \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_n := F_y \cdot Z_x$

The design flexure strength $\phi_b M_n := \phi_b \cdot M_n = 358\ kip \cdot ft$

The design flexure strength (Table 3-2) $\phi_b M_{px} := 358\ kip \cdot ft$

$$check := \textbf{if}\left(\phi_b M_n \ge M_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Check if $\phi_v V_{nx} \ge V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 53.6$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$

$$check := \textbf{if}\left(\lambda_w \le \lambda, \text{“YES”}, \text{“NO”}\right) = \text{“YES”}$$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 20.7\ in$

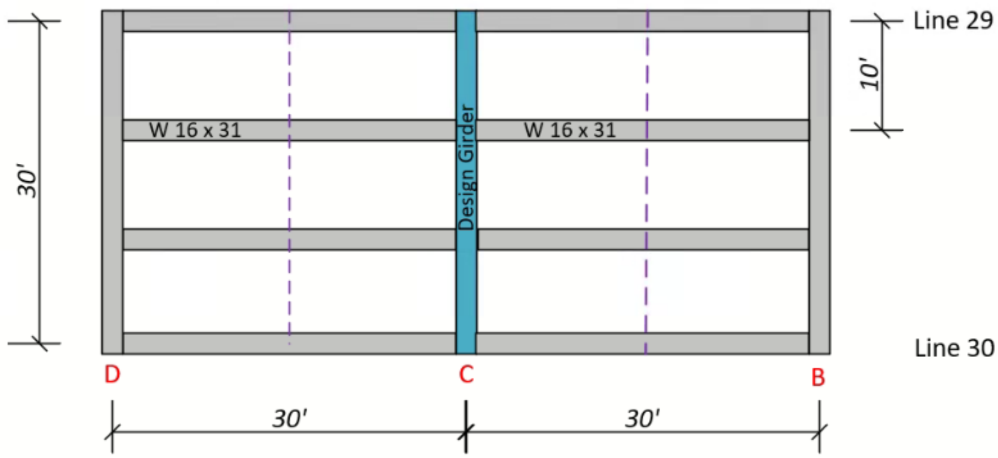
Web thickness of section (Table 1-1) $t_w := 0.35\ in$

Area of web (G2-1)	$A_w := d \cdot t_w = 7.245\ in^2$
The nominal shear strength (G2-1)	$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 217\ kip$
The design shear strength	$\phi_v V_n := \phi_v \cdot V_n = 217\ kip$
Selected section factored shear strength (Table 3-2)	$\phi_v V_{nx} := 217\ kip$

$$check := \text{if}\left(\phi_v V_{nx} \ge V_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

*Summary: Using the **W21X44** beam for this part 1 preliminary design. Assume the self-weight is **50 lb/ft** which is heavier than **W21X44**. Therefore, it is **not** necessary to refine the M_u & V_u. Using F-2 for the flexure strength calculation because **W21X44** has **compact** web and flanges according to Table 4-1.b **case 1& 15**. Be careful with the ϕ**=0.9**. Using the G1 & G2-1a for the shear strength calculation. Be careful with the ϕ**=1.0**. The validation of factored moment and shear strength using Table 3-2 in AISC is provided as above.*

Part 2: A simple-span *girder* at the 3rd floor, on column line C, spanning between column lines 29 and 30. Assume the girder is braced for positive moment only by the supported floor beams (no bracing provided by the deck).



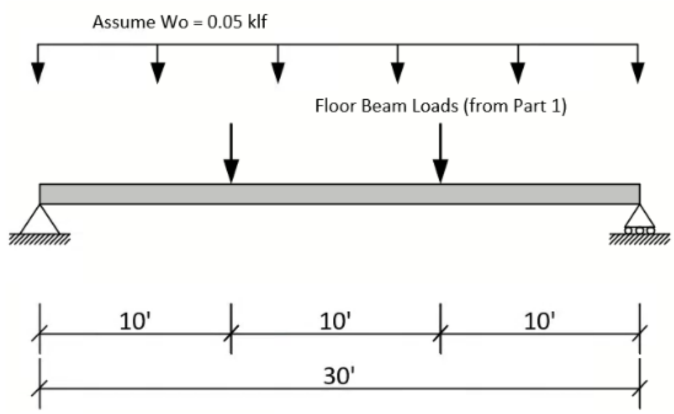
Recommend Design Steps for a beam with discrete lateral bracing ($L_b > 0$)

1. Compute factored moment and shear, M_u & V_u.Assume a beam self-weight (i.e., 50 or 100 lb/ft). Compute the associated C_b value. If possible, use **Table 3-1** for C_b.
2. Compute required I_x based on deflection limits. Be careful with units.
3. Use **Table 3-10**, select the lightest section. Note the $\phi_b M_{px}$ values in Table for F_y = 50 ksi. You will get into **Table 3-10** with L_b and $\frac{M_{u\_max}}{C_b}$.
4. If I_x for the trial section is smaller than the required I_x, refer to Table 3-3 and select the most economical section based on I_x. (Skip if $I_{x\_trial} \geq I_{x\_req}$)
5. Analyze the trial section to verify capacity

- Moment capacity (This is **highly necessary** regardless of the assumption of self-weight) (i.e., $C_b > 1.0$. $\phi_b M_{px}$ may be lower than M_u because the design capacity, ϕM_n, cannot increase beyond ϕM_{px}.
  Therefore, the trial section could fail even it "looks good". Therefore, we **need to check moment capacity** for **discretely laterally braced beams.** Do **iterations** if needed to determine the lightest section)
- Shear capacity (if F_y= 50 ksi, you can use the Table 3-2 and verify $\phi_v V_n$)
- Deflection limits (confirm final selection has Ix > Ix,req'd)

Determine the factored moment and shear, M_u & V_u (Refer to Part1 calculations)

According to figure shown above, the design girder is subjected point loads from floor beams from B to C, and C to D. Therefore, we need to determine the end shears for floor beams refer to Part 1 calculation process. Assume the **W21X44** was used for floor beams as same as Part 1. Assume the girder self-weight is 50 lb/ft.



Floor beam length $L := 30\ ft$

Unbraced length $L_b := \frac{L}{3} = 10\ ft$

Tributary width $W := 10\ ft$

Dead load on the floor beam $w_D := 75\ psf \cdot W = 0.75\ klf$

Live load on the floor beam $w_L := 100\ psf \cdot W = 1\ klf$

Self-weight of **W21X44** floor beam (Part 1) $w_O := 44\ plf = 0.044\ klf$

End support dead load shear (B to C) $V_{End\_D} := \left(w_D + w_O\right) \cdot \frac{L}{2} = 11.91\ kip$

End support live load shear (B to C) $V_{End\_L} := w_L \cdot \frac{L}{2} = 15\ kip$

Note: The shear will be considered as point loads on the girder. Double them as below to account for floor beams from C to D because the floor framing is the same as B to C.

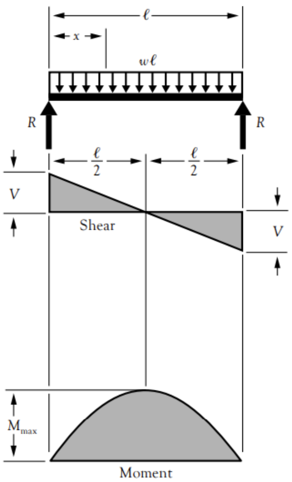
Point dead load on the girder $P_D := V_{End\_D} \cdot 2 = 23.82\ kip$

Point live load on the girder $P_L := V_{End\_L} \cdot 2 = 30\ kip$

Assume 50 lb/ft self-weight of the girder $w_O := 50\ plf = 0.05\ klf$

Note: Typically, we need to consider each segment for the girder because it has the unique L_b, C_b, and M_{u_max}. Therefore, each segment has a unique $\phi_b M_{px}$ and we need to compute all three segments. However, section 1 and section 3 have the same L_b, C_b, and M_{u_max}. Additionally, the middle segment can be reasoned to control without performing calculations. Because *Mu* is practically the same for all segments, but slightly higher in the middle segment because of uniformly distributed self-weight. *Lb* is the same for all segments. Cb is practically 1 for the middle segment, and much larger than 1 (about 1.67 for a nearly linear varying moment from 0 to max) for the end segments. Therefore, we need to consider **only one segment** for this case. For shears, we just use the **maximum shear** overall. Also, we use the **maximum deflection** along the beam which is the center for this case.

Figure 1 Simple Beam – Uniformly Distributed Load



$$R = V \quad \ldots\ldots\ldots\ldots \quad = \frac{w\ell}{2}$$

$$V_x \quad \ldots\ldots\ldots\ldots \quad = w\left(\frac{\ell}{2} - x\right)$$

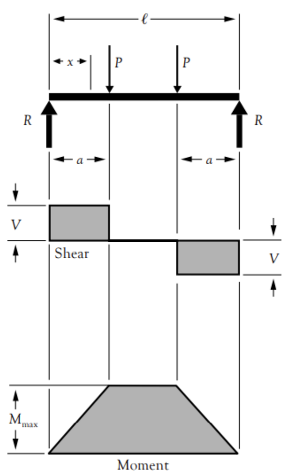
$$M_{max} \text{ (at center)} \quad \ldots\ldots\ldots \quad = \frac{w\ell^2}{8}$$

$$M_x \quad \ldots\ldots\ldots\ldots \quad = \frac{wx}{2}(\ell - x)$$

$$\Delta_{max} \text{ (at center)} \quad \ldots\ldots\ldots \quad = \frac{5w\ell^4}{384\,EI}$$

$$\Delta_x \quad \ldots\ldots\ldots\ldots \quad = \frac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$$

Figure 9 Simple Beam – Two Equal Concentrated Loads Symmetrically Placed



$$R = V \quad \ldots\ldots\ldots\ldots \quad = P$$

$$M_{max} \text{ (between loads)} \quad \ldots\ldots\ldots \quad = Pa$$

$$M_x \text{ (when } x < a) \quad \ldots\ldots\ldots \quad = Px$$

$$\Delta_{max} \text{ (at center)} \quad \ldots\ldots\ldots \quad = \frac{Pa}{24EI}(3\ell^2 - 4a^2)$$

$$\Delta_x \text{ (when } x < a) \quad \ldots\ldots\ldots \quad = \frac{Px}{6EI}(3\ell a - 3a^2 - x^2)$$

$$\Delta_x \left(\text{when } x > a \text{ and } < (\ell - a)\right) \quad \ldots \quad = \frac{Pa}{6EI}(3\ell x - 3x^2 - a^2)$$

Dead load moment
(unfactored, refer to above figures)

$$M_D(x) := \begin{array}{|l} \text{if } (x \le 10\ ft) \\ \quad \dfrac{(w_O \cdot x)}{2}(L - x) + P_D \cdot x \\ \text{if } (10\ ft < x \le 20\ ft) \\ \quad \dfrac{(w_O \cdot x)}{2}(L - x) + P_D \cdot 10\ ft \\ \text{if } (20\ ft < x \le 30\ ft) \\ \quad \dfrac{(w_O \cdot x)}{2}(L - x) + P_D \cdot (30\ ft - x) \end{array}$$

Live load moment
(unfactored, refer to above figures)

$$M_L(x) := \begin{array}{l} \text{if } (x \le 10\ ft) \\ \quad P_L \cdot x \\ \text{if } (10\ ft < x \le 20\ ft) \\ \quad P_L \cdot 10\ ft \\ \text{if } (20\ ft < x \le 30\ ft) \\ \quad P_L \cdot (30\ ft - x) \end{array}$$

Dead load shear
(unfactored, refer to above figures)

$$V_D := \frac{(w_O \cdot L)}{2} + P_D = 24.57\ kip$$

Live load shear
(unfactored, refer to above figures)

$$V_L := P_L = 30\ kip$$

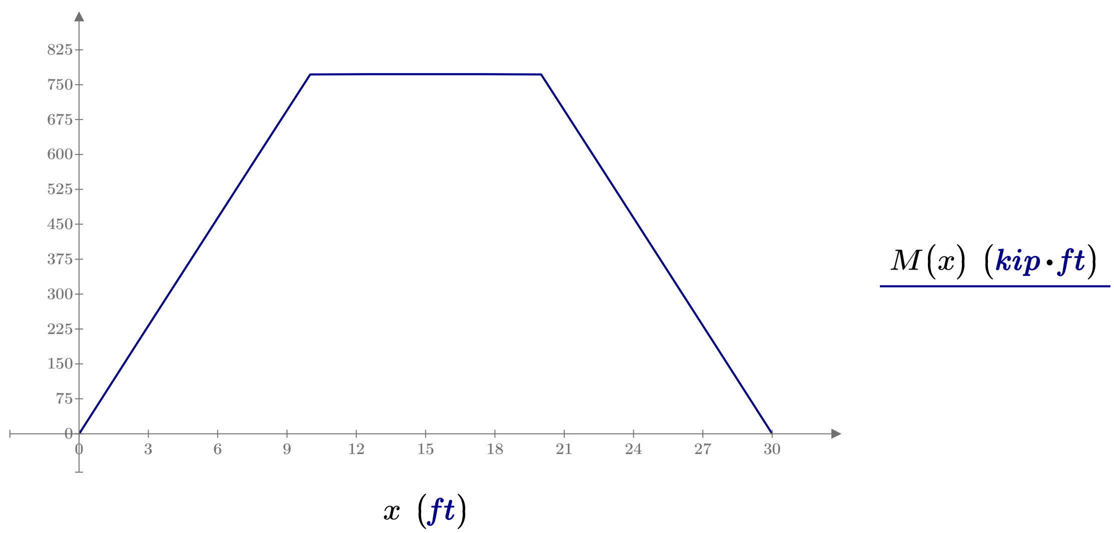
Load combination

1.2D + 1.6L load combination

$$M(x) := 1.2 \cdot M_D(x) + 1.6 \cdot M_L(x)$$

Range of x

$$x := 0, 1\ in .. L$$



Section 2-2 from x = 0 to 10 ft

$x := 12.5\ ft$ $\quad M_A := M(x) = 772.4\ kip \cdot ft$ $\quad x := 15\ ft$ $\quad M_B := M(x) = 772.59\ kip \cdot ft$

$x := 17.5\ ft$ $\quad M_C := M(x) = 772.4\ kip \cdot ft$ $\quad x := 15\ ft$ $\quad M_{max2} := M(x) = 772.59\ kip \cdot ft$

$$C_b := \frac{(12.5 \cdot M_{max2})}{2.5 \cdot M_{max2} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 1.0001$$

Factored moment $M_u := M_{max2} = 772.59\ kip \cdot ft$

Factored shear $V_u := 1.2 \cdot V_D + 1.6 \cdot V_L = 77.48\ kip$

Determine the required moment of inertia, I_x

Modulus of elasticity of steel $E := 29000\ ksi$

Max allowable deflection at the mid for live load $\Delta_{L\_max} := \dfrac{L}{360} = 1\ in$

Max allowable deflection at the mid for total load $\Delta_{T\_max} := \dfrac{L}{240} = 1.5\ in$

Note: The maximum deflection along the beam is located at the middle of the girder.

Required moment of inertia of live load $I_{x\_L} := \dfrac{\left(P_L \cdot L_b\right)}{24 \cdot E \cdot \Delta_{L\_max}} \cdot \left(3 \cdot L^2 - 4 \cdot {L_b}^2\right) = 1713.103\ in^4$

Required moment of inertia of total load $I_{x\_T} := \dfrac{\left(5 \cdot w_O \cdot L^4\right) + 16 \cdot \left(P_D + P_L\right) \cdot L_b \cdot \left(3 \cdot L^2 - 4 \cdot {L_b}^2\right)}{384 \cdot E \cdot \left(\Delta_{T\_max}\right)} = 2069.82\ in^4$

Required moment of inertia (max) $I_{x\_req} := \max\left(I_{x\_L}, I_{x\_T}\right) = 2069.82\ in^4$

Use Table 3-10, select the lightest section regarding to L_b ***and*** M_{u_eff}

Unbraced length $L_b := \dfrac{L}{3} = 10\ ft$

Effective factored moment M_{u_eff} $M_{u\_eff} := \dfrac{M_u}{C_b} = 772.5\ kip \cdot ft$

Note: Try the first right and upper bolded (lightest in group) line as the order below with L_b = 10 ft and M_{u_eff} = 772.5 kip*ft.

General process shown as below table is:
check **Table 3-10** to ensure $\phi_b M_n > M_{u\_eff}$
check **Table 3-2** to ensure $\phi_b M_{px} > M_u$
if both of those are satisfied, the shape will work for strength.
If serviceability is also considered (as in this assignment), then also check **Table 3-3** to ensure $I_{x\_trial} \geq I_{x\_req}$
When all those criteria are satisfied, then proceed to verify the capacity using the Specifications.

Trial section	Mu(kip-ft)	φbMpx(kip-ft)	Ix,min(in^4)	Ix(in^4)	
W27X84	767.91	915	2054.973	2850	Worth to try
W30X90	767.91	1060	2054.973	3610	
W30X99	767.91	1170	2054.973	3990	

Try **W27X84** (bolded on page 3-26) with $\phi_b M_{px} = 915$ kip-ft.

Analyze the trial section to verify capacity

Check if $\phi_b M_n \geq M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 2.07\ in$

Yield stress $F_y := 50\ ksi$

L_p Pg. 16-48. (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.312\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 2.81\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 2.54\ in$

Elastic section modulus Table 1-1 $S_x := 213\ in^3$

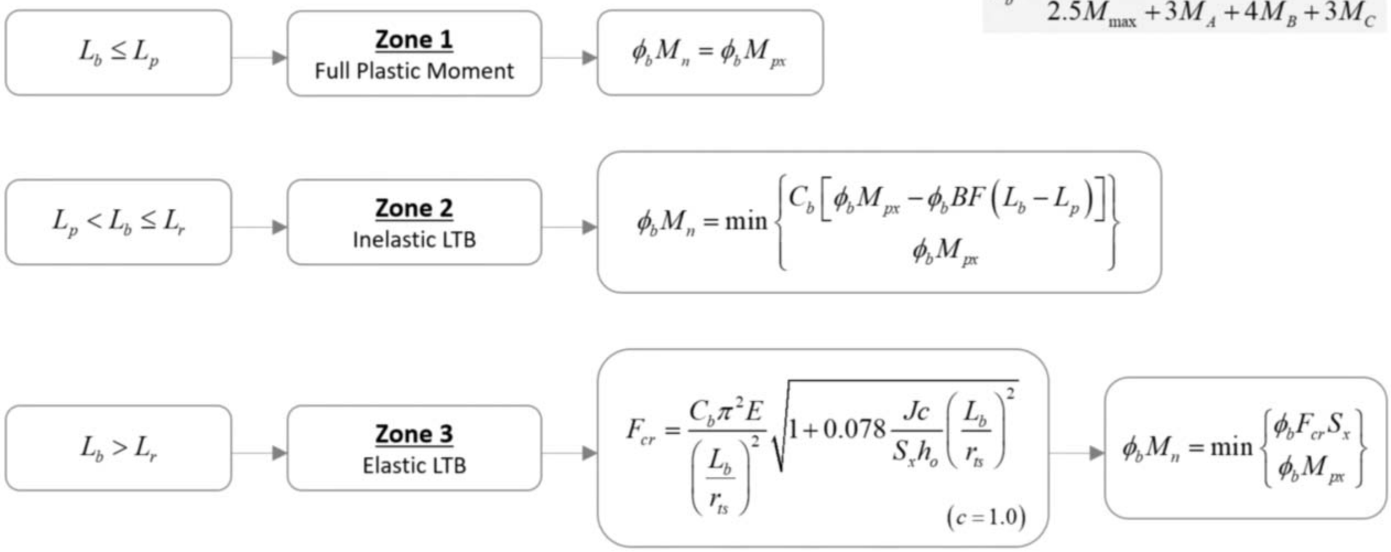
Distance between flanges centroids Table 1-1 $h_o := 26.1\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 20.757\ ft$$

Flowchart of AISC Equations for LTB Capacity

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$$

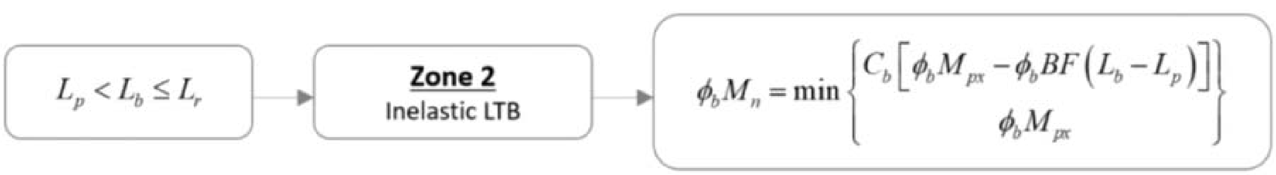


$check := \textbf{if}\left(L_b \le L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \textbf{if}\left(L_p \le L_b \le L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Inelastic LTB”}$

$check := \textbf{if}\left(L_r \le L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Not Elastic LTB”}$

Note: Therefore, it is in the **Zone 2** range (**inelastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Modulus of elasticity E $\qquad E := 29000\ ksi$

Plastic section modulus (Table 1-1) $\qquad Z_x := 244\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\qquad \lambda_f := 7.78$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$check := \text{if}\left(\lambda_f \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 52.7$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$check := \text{if}\left(\lambda_w \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 1016.67\ \boldsymbol{kip \cdot ft}$

Factored plastic moment
(confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 915\ \boldsymbol{kip \cdot ft}$

BF term shown in F2-2 $BF := \frac{\left(M_p - 0.7 \cdot F_y \cdot S_x\right)}{L_r - L_p} = 29.41\ \boldsymbol{kip}$

Factored moment capacity (F2-2) $\phi_b M_n := min\left(C_b \cdot \left(\phi_b \cdot M_p - \phi_b \cdot BF \cdot \left(L_b - L_p\right)\right), \phi_b \cdot M_p\right) = 843.943\ \boldsymbol{kip \cdot ft}$

$check := \text{if}\left(\phi_b M_n \ge M_u, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Check if $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 2850\ in^4$

$$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Check if $\phi_v V_{nx} \geq V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 52.7$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$$check := \text{if}\left(\lambda_w \leq \lambda, \text{"YES"}, \text{"NO"}\right) = \text{"YES"}$$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 26.7\ in$

Web thickness of section (Table 1-1) $t_w := 0.46\ in$

Area of web (G2-1) $A_w := d \cdot t_w = 12.282\ in^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 368\ kip$

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 368\ kip$

Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 368\ kip$

$$check := \text{if}\left(\phi_v V_{nx} \geq V_u, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Note: We assumed the girder weight is 50 lb/ft to determine the M_u **&** V_u. But we actually use **W27X84** for the final design which is **heavier** than we assumed. Therefore, we need to recalculate the M_u **&** V_u based on the **84 lb/ft** self-weight and **verify capacity again**. **Only check moment** for this case, because the **shear and moment of inertia** would not be a problem for this little self-weight change.

Factored moment (with new self-weight) $M_{u\_new} := 777.2\ kip \cdot ft$

$$check := \text{if}\left(\phi_b M_n \geq M_{u\_new}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Summary: Using the ***W27X84*** *beam for this part 2 preliminary design. Assume the self-weight is* ***50 lb/ft*** *which is lighter than* ***W27X84****. Therefore, "new"* M_u *associated with W27X84 was checked. Using F-2 for the flexure strength calculation because* ***W27X84*** *has* ***compact*** *web and flanges according to Table 4-1.b* ***case 10 &15****. Be careful with the* ϕ***=0.9****. Using the G1 & G2-1a for the shear strength calculation. Be careful with the* ϕ***=1.0****. The validation of factored moment and shear strength using Table 3-2 in AISC is provided above.*

Part 3: Redesign the girder on column line C, spanning between column lines 29 and 30, assuming that moment connections are provided at both ends of the beam. For simplicity in this preliminary design, assume that the floor framing is identical for the adjacent bays opposite the column at each end of the girder. Also, neglect stiffness contributions and moment distributions to the columns. Analyze the girder as a **3-span continuous beam**. Design to accommodate pattern loading:
(1) live load only on the bay from column lines 29 to 30;
(2) live load only on the adjacent bays;
(3) live load on the bay from column lines 29 to 30 and one adjacent bay; and
(4) live load only on one adjacent bay.
Assume the girder is **braced for positive moment only by the supported floor beams** (no bracing provided by the deck). Assume the girder is braced for both positive and negative moment at supports (columns).

Note: The design procedures are the same as in Part 2. The dead and live point loads on the girder from floor beams are calculated in Part 2. **Assume the girder self-weight is 50 lb/ft**. There total four loading scenarios are considered here.

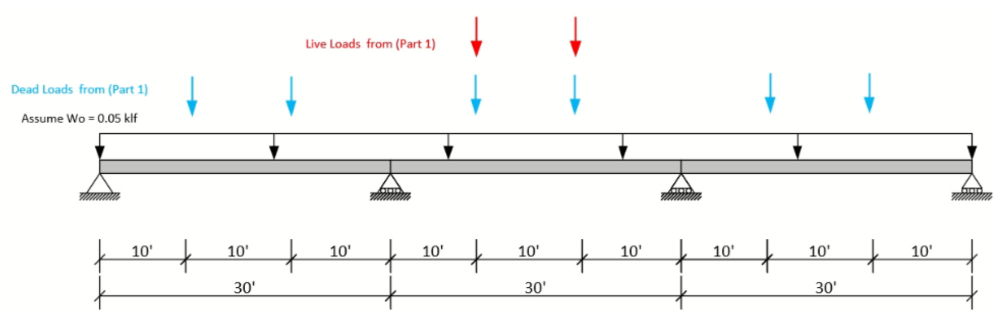
The point loads from floor beams (refer to Part 2)

Point dead load on the girder $P_D := 23.82\ \boldsymbol{kip}$

Point live load on the girder $P_L := 30\ \boldsymbol{kip}$

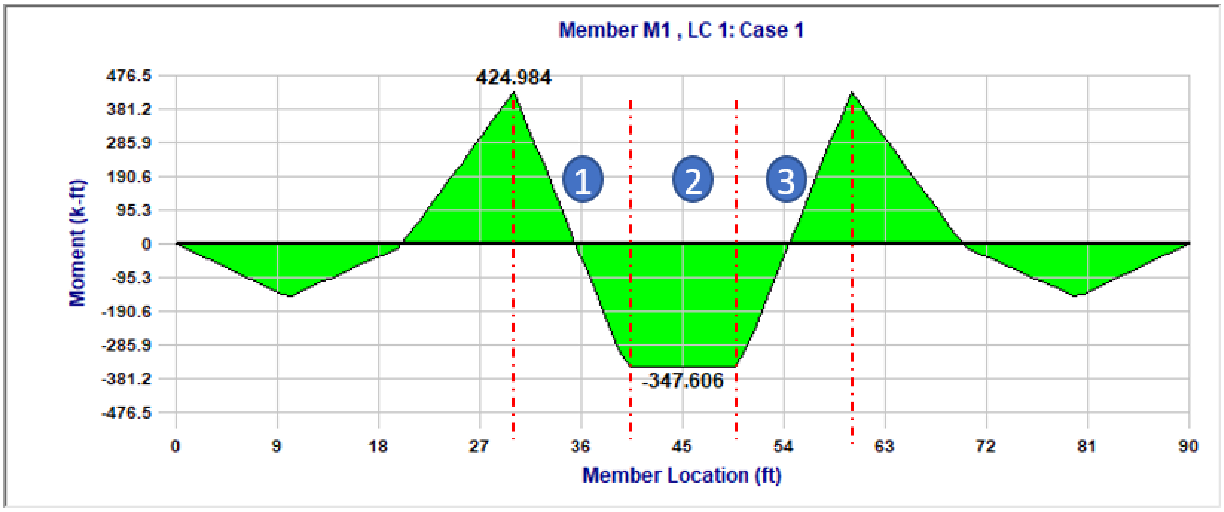
Determine the factored moment and shear, M_u & V_u (total four load cases)

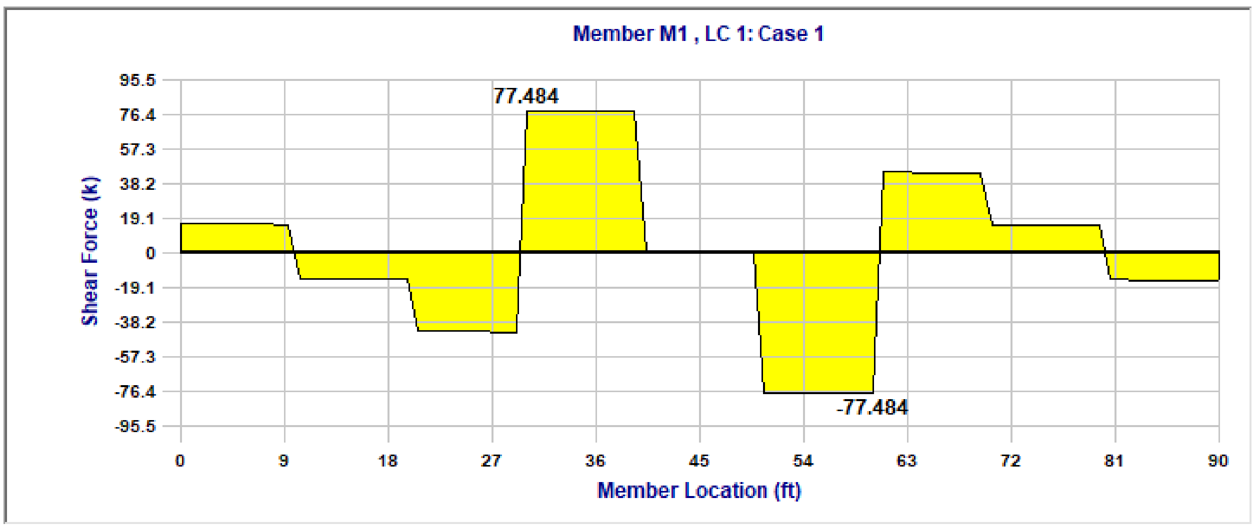
Case (1) live load only on the bay from column lines 29 to 30



Note: We need to use the *Mu* diagram to determine the *Cb* for each load case. RISA 2D was used to get the moment, shear, and deflection diagram for the **1.2D + 1.6L load combination and D+L for serviceability check.** Note that **RISA used a convention** where moments are drawn on the tension side. This graphically results in moment diagrams flipped from our typical convention (moments drawn on compression side). Therefore, the floor beams brace the girder when RISA indicates a *negative* moment, such as at the midspan of the girder we're currently designing. By our convention, the end moments for the design span are negative (and braced because at supports), and the moments where beams frame in are positive (and therefore also braced, because transverse framing restrains the girder when its top is in compression - where the transverse framing attaches).

1.2D + 1.6L Mu and Vu diagrams





Note: We need to consider each segment for the girder because it has the unique L_b, C_b, and M_{u_max}. Therefore, each segment has a unique $\phi_b M_{px}$ and we need to compute all three segments. However, section 1 and section 3 have the same L_b, C_b, and M_{u_max}. Therefore, we need to consider **two segments** (30 to 40 and 40 to 50 ft) for this case.

Section 1-1 from x = 30 to 40 ft

$x := 32.5\ ft$ $\quad M_A := -232.1\ kip \cdot ft$ $\qquad x := 35\ ft$ $\quad M_B := -38.9\ kip \cdot ft$

$x := 37.5\ ft$ $\quad M_C := 153.9\ kip \cdot ft$ $\qquad x := 30\ ft$ $\quad M_{max} := -425\ kip \cdot ft$

$$C_{b1} := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 2.2358$$

Compute M_{u_eff} for section 1 as shown above

Maximum factored moment in (section 1) $M_{u1} := \text{abs}\left(M_{max}\right) = 425 \ \textit{kip} \cdot \textit{ft}$

Effective factored moment M_{u_eff} $M_{u\_eff1} := \dfrac{M_{u1}}{C_{b1}} = 190.088 \ \textit{kip} \cdot \textit{ft}$

Section 2-2 from x = 40 to 50 ft

$x := 42.5 \ \textit{ft}$ $M_A := 346.8 \ \textit{kip} \cdot \textit{ft}$ $x := 45 \ \textit{ft}$ $M_B := 347.6 \ \textit{kip} \cdot \textit{ft}$

$x := 47.5 \ \textit{ft}$ $M_C := 346.8 \ \textit{kip} \cdot \textit{ft}$ $x := 45 \ \textit{ft}$ $M_{max} := 347.6 \ \textit{kip} \cdot \textit{ft}$

$$C_{b2} := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 1.0011$$

Compute M_{u_eff} for section 2 as shown above

Maximum factored moment in (section 2) $M_{u2} := \text{abs}\left(M_{max}\right) = 347.6 \ \textit{kip} \cdot \textit{ft}$

Effective factored moment M_{u_eff} $M_{u\_eff2} := \dfrac{M_{u2}}{C_{b2}} = 347.216 \ \textit{kip} \cdot \textit{ft}$

Determine the Vu for the Case 1 $V_u := 77.484 \ \textit{kip}$

Determine the required moment of inertia, I_x

Length of the design girder $L := 30 \ \textit{ft}$

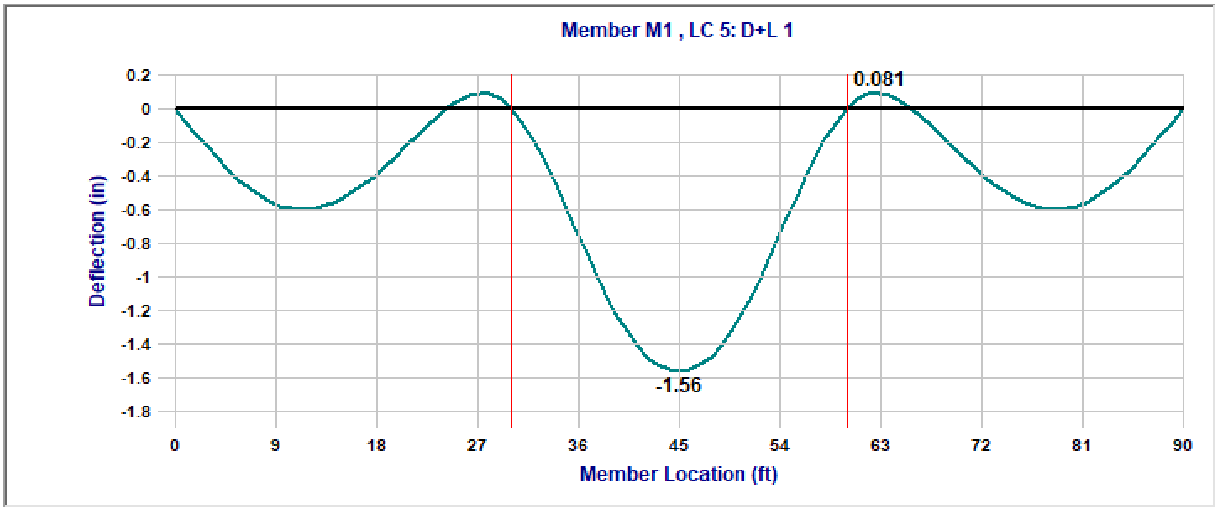
Modulus of elasticity of steel $E := 29000 \ \textit{ksi}$

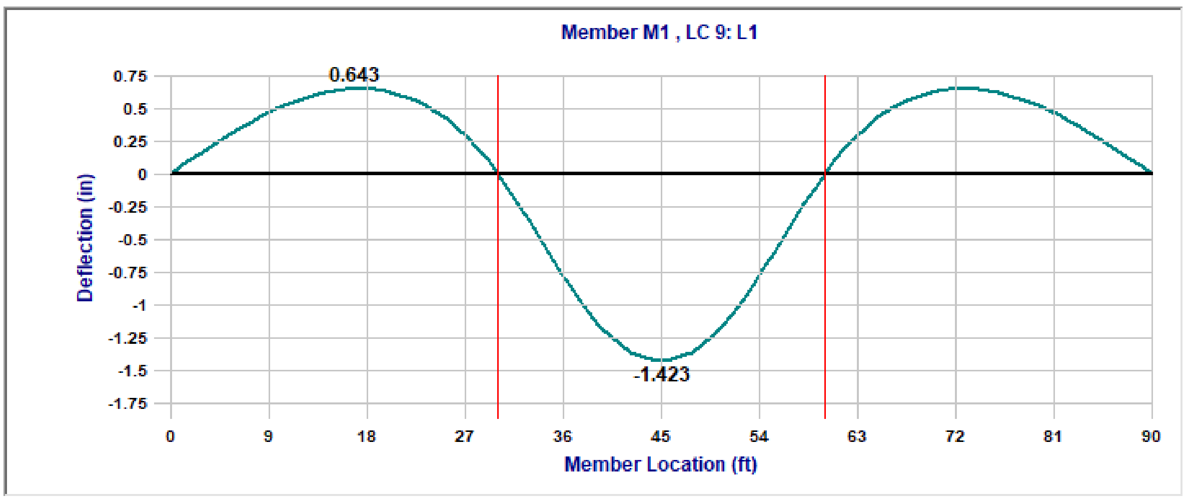
Max allowable deflection at the mid for total load $\Delta_{T\_max} := \dfrac{L}{240} = 1.5 \ \textit{in}$

Max allowable deflection at the mid for live load $\Delta_{L\_max} := \dfrac{L}{360} = 1 \ \textit{in}$

Note: The maximum deflection is located at the middle of the girder when live load only on the bay from column lines 29 to 30 among four load cases. I used **W18X50 (*not for the design*)** in the models, therefore, we can get the **required moment of inertia** based on the deflection diagrams. The moment of inertia is inversely proportional to the deflection.

D+L(Total) and only L deflection diagrams for W18X50 (*Ix = 800 in ^4*)





Ix of **W18X50** $I_{x\_trial} := 800 \; in^4$

Total deflection (above 1st diagram) $\Delta_T := 1.56 \; in$

Live load deflection (above 2nd diagram) $\Delta_L := 1.423 \; in$

Required moment of inertia of total load $I_{x\_T} := \dfrac{\Delta_T}{\Delta_{T\_max}} \cdot I_{x\_trial} = 832 \; in^4$

Required moment of inertia of live load $I_{x\_L} := \dfrac{\Delta_L}{\Delta_{L\_max}} \cdot I_{x\_trial} = 1138.4 \; in^4$

Required moment of inertia (max) $I_{x\_req} := \max\left(I_{x\_L}, I_{x\_T}\right) = 1138.4 \; in^4$

Use Table 3-10, select the lightest section regarding to L_b *and* M_{u_eff}

Unbraced length for both **Section 1 & 2** $L_b := 10\ ft$

Max factored moment M_{u_eff}
(section 1: M_u **governs)** $M_u := \max\left(M_{u1}, M_{u2}\right) = 425\ kip \cdot ft$

Effective factored moment M_{u_eff}
(section 2: M_{u_eff} **governs)** $M_{u\_eff} := \max\left(M_{u\_eff1}, M_{u\_eff2}\right) = 347.216\ kip \cdot ft$

Note: For design, pick the segment with the largest Mu,max/Cb or the largest Lb. Find the right and upper bolded (lightest in group) line as the order below with L_b **= 10 ft and** M_{u_eff} **= 347.2 kip*ft.**
General process shown as below table is:
check **Table 3-10** to ensure $\phi_b M_n > M_{u\_eff}$
check **Table 3-2** to ensure $\phi_b M_{px} > M_u$
if both of those are satisfied, the shape will work for strength.
If serviceability is also considered (as in this assignment), then also check **Table 3-3** to ensure $I_{x\_trial} \geq I_{x\_req}$
When all those criteria are satisfied, then proceed to verify the capacity using the Specifications.

Selection process: Using Table 3-10 and find the first try section **W18X55** on Pg. 3-117, but the $\phi_b M_{px}$ is smaller than Mu (**not good**). This is not the correct process, but we could try to then find the right and upper bolded (lightest in group) line is **W21X55** with $\phi_b M_{px}$ =473 kip*ft (Table 3-2). Next, we need then verify its $\phi_b M_n$ regarding to LTB limit state when Lb =10 ft (**not good as shown in Pg. 3-117**). Then Find the **W21X62** with $\phi_b M_n$ (for LTB) larger than Mu = 425 kip*ft and $\phi_b M_{px}$ = 540 kip*ft. The moment of inertia of **W21X62** is also larger than Ix,required.
Therefore, **W21X62** is the first section you may need to try.

Trial section	Mu(kip-ft)	φbMpx(kip-ft)	φ Mn(LTB)	φ Mn(min)	Ix,min(in^4)	Ix(in^4)	
W18X55	425	420			1138.4		No
W21X55	425	472.5	405		1138.4		No
W21X62	425	540	? > 425	?	1138.4	1330	Worth to try

Analyze the trial section to verify capacity

Check if $\phi_b M_n \geq M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 1.77\ in$

Yield stress $F_y := 50\ ksi$

L_p Pg. 16-48. (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 6.252\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 1.83\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 2.15\ in$

Elastic section modulus Table 1-1 $S_x := 127\ in^3$

Distance between flanges centroids Table 1-1 $h_o := 20.4\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

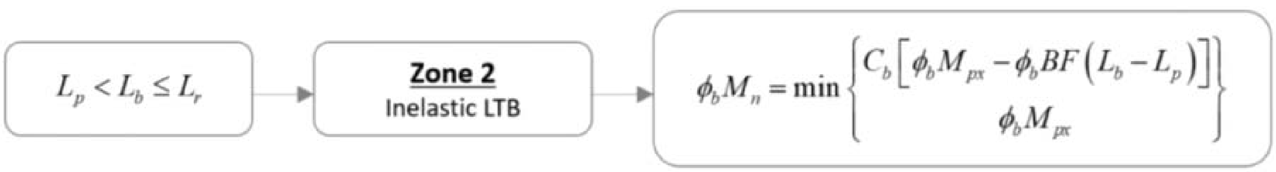
$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 18.131\ ft$$

$check := \textbf{if}\left(L_b \le L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \textbf{if}\left(L_p \le L_b \le L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Inelastic LTB”}$

$check := \textbf{if}\left(L_r \le L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Not Elastic LTB”}$

Note: Therefore, it is in the **Zone 2** range (**inelastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Modulus of elasticity E $E := 29000\ ksi$

Plastic section modulus (Table 1-1) $Z_x := 144\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 6.70$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$check := \textbf{if}\left(\lambda_f \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 46.9$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\dfrac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\dfrac{E}{F_y}} = 137.274$

$$check := \text{if}\left(\lambda_w \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 600\ \boldsymbol{kip \cdot ft}$

Factored plastic moment
(confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 540\ \boldsymbol{kip \cdot ft}$

BF term shown in F2-2 $BF := \dfrac{\left(M_p - 0.7 \cdot F_y \cdot S_x\right)}{L_r - L_p} = 19.33\ \boldsymbol{kip}$

Factored moment capacity (F2-2)

Moment capacity for Segment 1 $\phi_b M_{n1} := min\left(C_{b1} \cdot \left(\phi_b \cdot M_p - \phi_b \cdot BF \cdot \left(L_b - L_p\right)\right), \phi_b \cdot M_p\right) = 540\ \boldsymbol{kip \cdot ft}$

Moment capacity for Segment 2 $\phi_b M_{n2} := min\left(C_{b2} \cdot \left(\phi_b \cdot M_p - \phi_b \cdot BF \cdot \left(L_b - L_p\right)\right), \phi_b \cdot M_p\right) = 475.3\ \boldsymbol{kip \cdot ft}$

$$check := \text{if}\left(\phi_b M_{n1} \ge M_{u1}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$check := \text{if}\left(\phi_b M_{n2} \ge M_{u2}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Note: We need two checks: Segment 1 with Cb > 1 vs Mu1 and Segment 2 with Cb = 1 vs Mu2.

Check if $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 1330\ in^4$

$$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Check if $\phi_v V_{nx} \geq V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 46.9$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$

$$check := \text{if}\left(\lambda_w \leq \lambda, \text{"YES"}, \text{"NO"}\right) = \text{"YES"}$$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 21\ in$

Web thickness of section (Table 1-1) $t_w := 0.40\ in$

Area of web (G2-1) $A_w := d \cdot t_w = 8.4\ in^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 252\ kip$

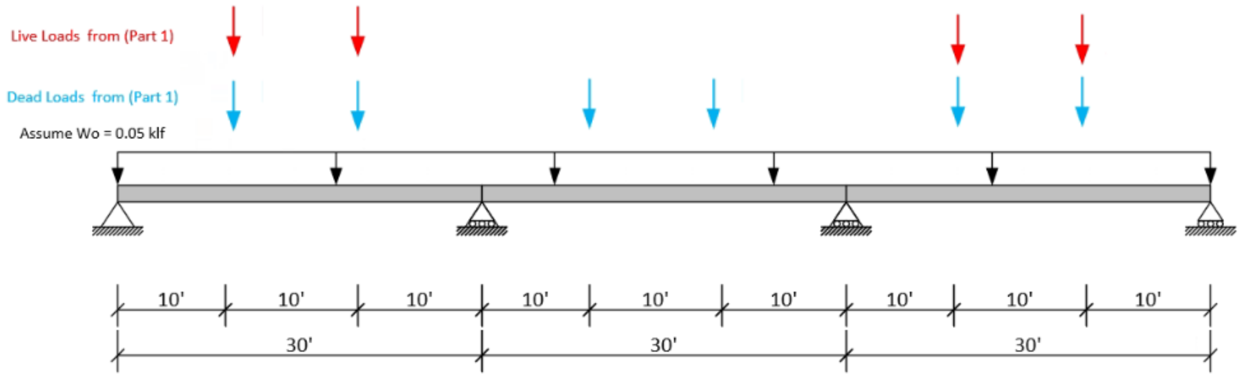
The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 252\ kip$

Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 252\ kip$

$$check := \text{if}\left(\phi_v V_{nx} \geq V_u, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

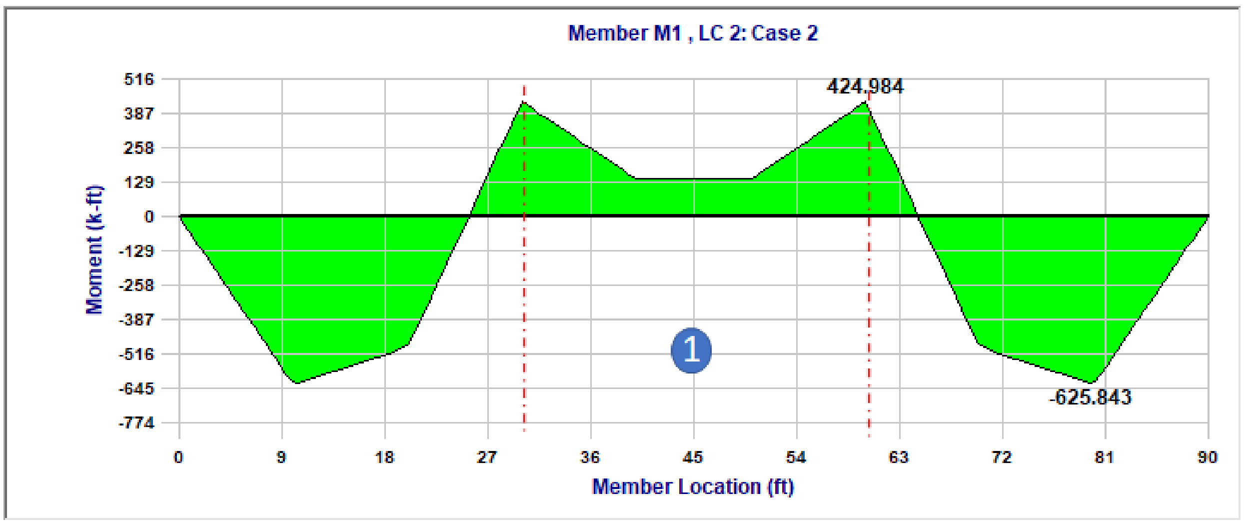
Note: We assumed the girder weight is 50 lb/ft to determine the M_u **&** V_u**.** But we actually use **W21X62** for the final design which is **heavier** than we assumed. The moment, shear and moment of inertia **would not be a problem for this little self-weight change**. Therefore, using the **W21X62** for case 1.

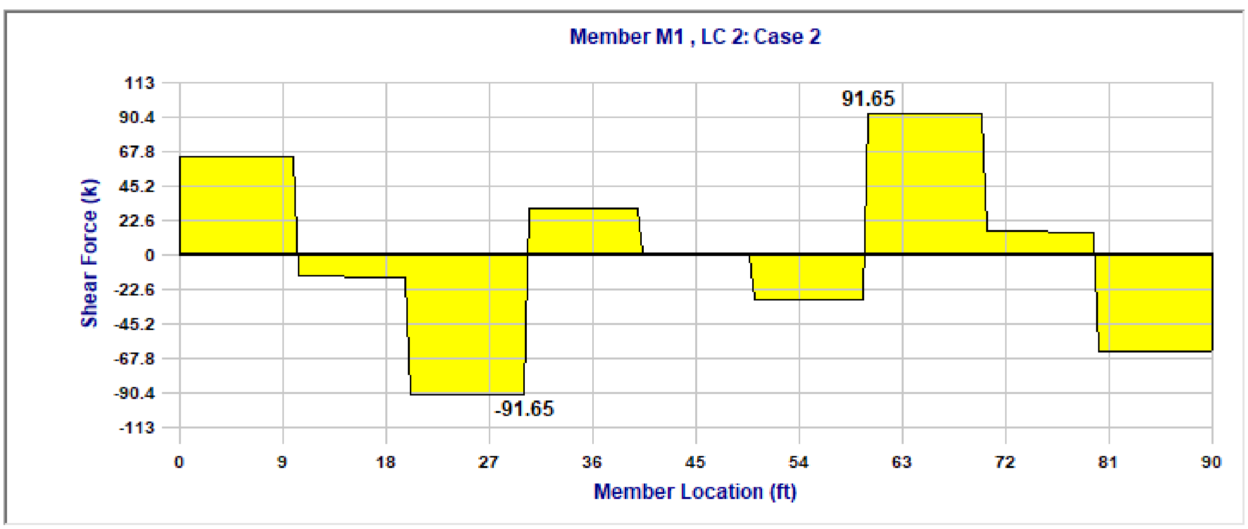
Case (2) live load only on the adjacent bays



Note: We need to use the *Mu* diagram to determine the *Cb* for each load case. RISA 2D was used to get the moment and shear diagram for the **1.2D + 1.6L load combination.** Recall that the Case 1 would govern for deflection checks, therefore, no need to check serviceability here.

1.2D + 1.6L Mu and Vu diagrams





Note: We only need to consider **one segment (30 to 60 ft)** for the girder here. The Lb for the negative moment is 30 ft for this case.(Recall: RISA uses an opposite moment convention to ours, so what it shows as "positive" has tension on the top and compression on the bottom of the girder. Floor framing attaches to the top of the girder and is assumed not to effectively restrain the bottom flange from lateral buckling.)

Section 1-1 from x = 30 to 60 ft

$x := 37.5\ ft$ $\quad M_A := -206.1\ kip \cdot ft$ $\quad x := 45\ ft$ $\quad M_B := -133\ kip \cdot ft$

$x := 52.5\ ft$ $\quad M_C := -206.1\ kip \cdot ft$ $\quad x := 30\ ft$ $\quad M_{max} := -425\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 1.8765$$

Compute M_{u_eff} for section 1 as shown above

Maximum factored moment in (section 1) $\qquad M_{u1} := \text{abs}\left(M_{max}\right) = 425\ kip \cdot ft$

Effective factored moment M_{u_eff} $\qquad M_{u\_eff1} := \dfrac{M_{u1}}{C_b} = 226.488\ kip \cdot ft$

Use Table 3-10, select the lightest section regarding to L_b and M_{u_eff}

Unbraced length for both **Section 1** $\qquad L_b := 30\ ft$

Max factored moment M_{u_eff} $\qquad M_u := M_{u1} = 425\ kip \cdot ft$

Effective factored moment M_{u_eff} $\qquad M_{u\_eff} := M_{u\_eff1} = 226.488\ kip \cdot ft$

Note: Find the right and upper bolded (lightest in group) line as the order below with L_b = 30 ft and M_{u_eff} = 226.49 kip*ft.

General process shown as below table is:
check **Table 3-10** to ensure $\phi_b M_n > M_{u\_eff}$
check **Table 3-2** to ensure $\phi_b M_{px} > M_u$
if both of those are satisfied, the shape will work for strength.
If serviceability is also considered (as in this assignment), then also check **Table 3-3** to ensure $I_{x\_trial} \geq I_{x\_req}$
When all those criteria are satisfied, then proceed to verify the capacity using the Specifications.

Selection process: Firstly, check if Case 1 section W21X62 works for the Case 2. Due to the increased Lb to 30 ft, the $\phi_b M_n$ **for LTB** was reduced to **291 kip*ft** which is not good**.** Then, we need using Table 3-10 and find the first try section **W12X65** on **Pg. 3-120**, but the $\phi_b M_{px}$ is smaller than *Mu* (**not good**). Dashes lines are less economical than solid lines in Table 3-10 *only for Cb = 1*. With Cb > 1, solid lines may not be adequate with low phi*Mp, but a dashed line may have the necessary plastic strength and so be the most efficient selection. Then find the **W16X67** with $\phi_b M_{px}$ = 488 kip*ft **(Table 3-2).** But the moment of inertia of **W16X67 (Table 1-1)** is smaller than *Ix, required*. You can find the moment of inertias of **W14X68, W12X72 and W14X74** on **Pg. 3-120** are also smaller than required or you may use **Table 3-3** to search the sections with *Ix >= Ix,required*.
Then you can find the W18X76 is a good trial section as shown in below table.

Trial section	Mu(kip-ft)	φbMpx(kip-ft)	φ Mn(LTB)	φ Mn(min)	Ix,min(in^4)	Ix(in^4)	
W21X62	425	540	291.1		1138.4		No
W12X65	425	356			1138.4		No
W16X67	425	488	> 425?		1138.4	954	No
W14X68	425	431	> 425?		1138.4	722	No
W12X72	425	405			1138.4	597	No
W14X74	425	473	> 425?		1138.4	795	No
W18X76	425	611	> 425?		1138.4	1330	Worth to try

Analyze the trial section to verify capacity

Check if $\phi_b M_n \ge M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 2.61\ in$

Yield stress $F_y := 50\ ksi$

L_p Pg. 16-48. (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 9.219\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 2.83\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 3.02\ in$

Elastic section modulus Table 1-1 $S_x := 146\ in^3$

Distance between flanges centroids Table 1-1 $h_o := 17.5\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

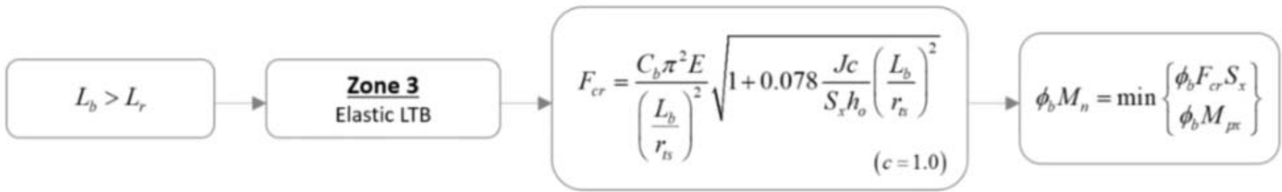
$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 27.08\ ft$$

$check := \mathbf{if}\left(L_b \le L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \textbf{if}\left(L_p \le L_b \le L_r, \text{"Inelastic LTB"}, \text{"Not Inelastic LTB"}\right) = \text{"Not Inelastic LTB"}$

$check := \textbf{if}\left(L_r \le L_b, \text{"Elastic LTB"}, \text{"Not Elastic LTB"}\right) = \text{"Elastic LTB"}$

Note: Therefore, it is in the **Zone 3** range (**elastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Modulus of elasticity E $E := 29000\ ksi$

Plastic section modulus (Table 1-1) $Z_x := 163\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 8.11$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$check := \textbf{if}\left(\lambda_f \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 37.8$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$check := \textbf{if}\left(\lambda_w \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$

Note: Both flange and web elements are compact, therefore using Equation F2-4 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 679.17\ \boldsymbol{kip \cdot ft}$

Factored plastic moment
(confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 611.25\ \boldsymbol{kip \cdot ft}$

BF term shown in F2-4
$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{(J \cdot c)}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 56.41\ \boldsymbol{ksi}$$

Note: We are allowed to use Fcr > Fy and multiply by the elastic section modulus for the ELTB check, because these moments calculated with very large Fcr values will always be limited by plastic moment, Fy * Zx.

Factored moment capacity (F2-3) $\phi_b M_n := min\left(\phi_b \cdot F_{cr} \cdot S_x, \phi_b \cdot M_p\right) = 611.25\ \boldsymbol{kip \cdot ft}$

$check := \textbf{if}\left(\phi_b M_n \ge M_u, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Check if $I_{x\_trial} \ge I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 1330\ \boldsymbol{in}^4$

$check := \textbf{if}\left(I_{x\_trial} \ge I_{x\_req}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Check if $\phi_v V_{nx} \ge V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 37.8$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$

$check := \textbf{if}\left(\lambda_w \le \lambda, \text{"YES"}, \text{"NO"}\right) = \text{"YES"}$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 18.2\ in$

Web thickness of section (Table 1-1) $t_w := 0.425\ in$

Area of web (G2-1) $A_w := d \cdot t_w = 7.735\ in^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 232\ kip$

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 232\ kip$

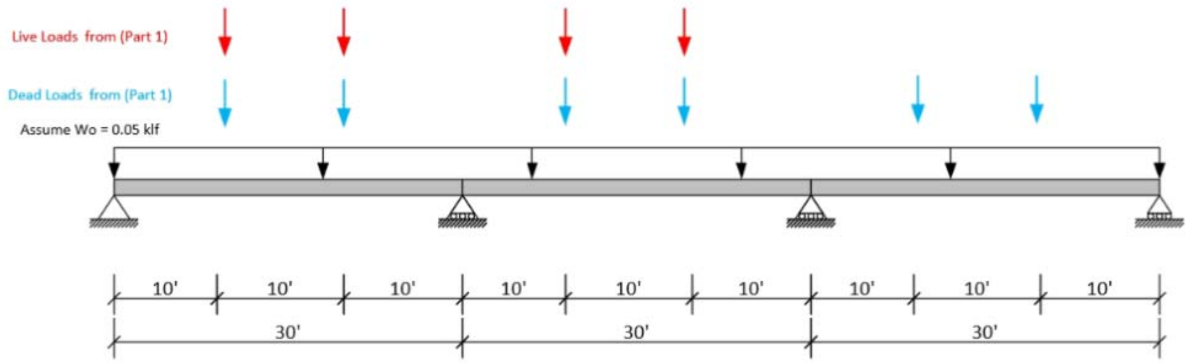
Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 232\ kip$

$$check := \text{if}\left(\phi_v V_{nx} \ge V_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Note: We assumed the girder weight is 50 lb/ft to determine the M_u **&** V_u**.** But we actually use **W18X76** for the final design which is **heavier** than we assumed. We use **W18X76** which has around 600 kip*ft moment capacity that is much larger than the required *Mu*. Therefore, the moment, shear and moment of inertia **would not be a problem for this little self-weight change**.

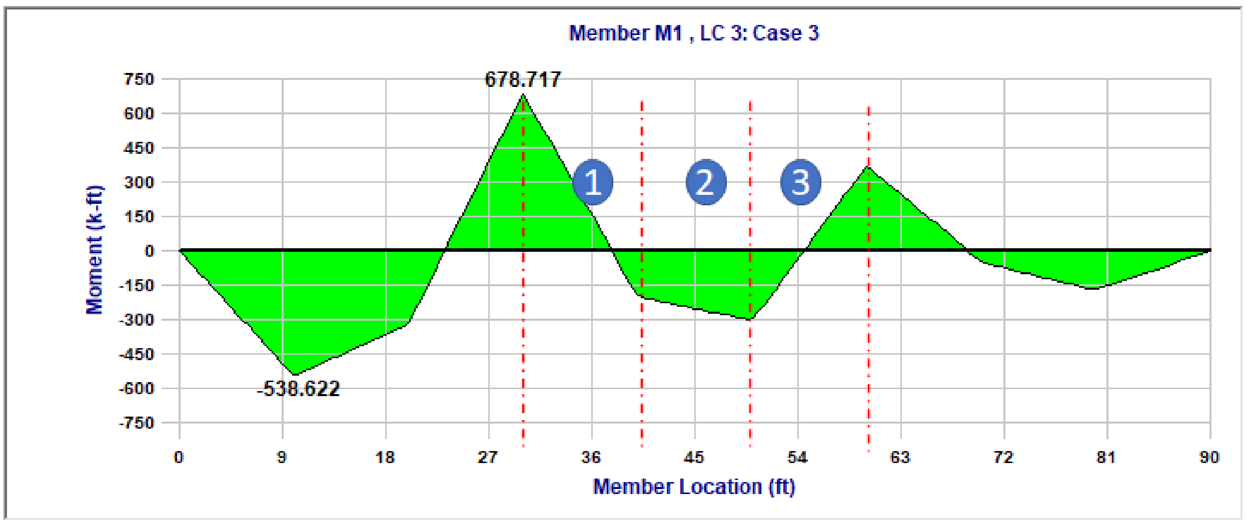
Therefore, using the **W18X76** for case 2.

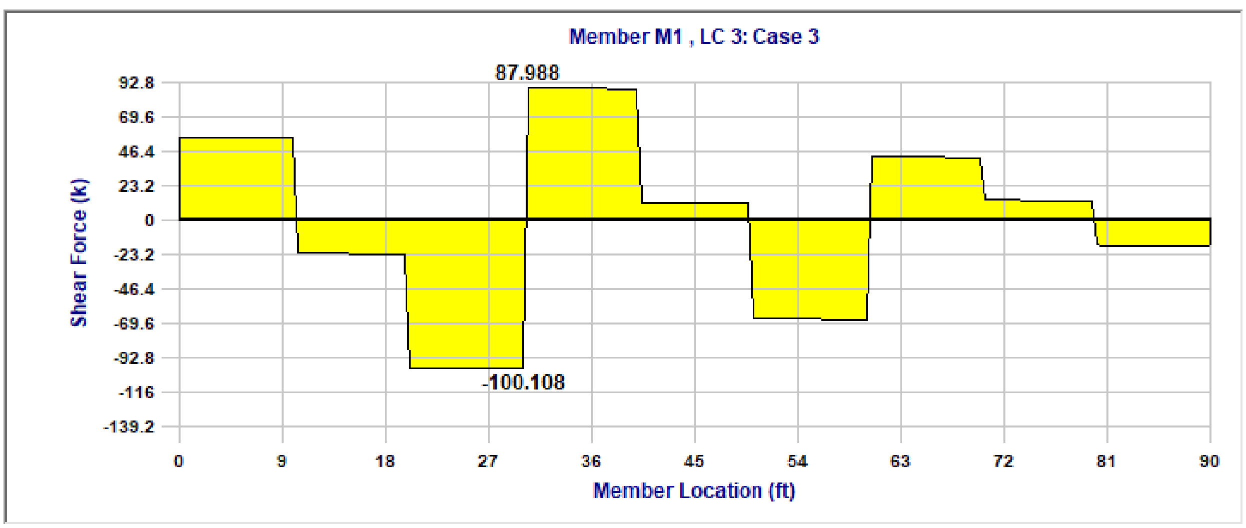
Case (3) live load on the bay from column lines 29 to 30 and one adjacent bay



Note: We need to use the *Mu* diagram to determine the *Cb* for each load case. RISA 2D was used to get the moment and shear diagram for the **1.2D + 1.6L load combination.** Recall that the Case 1 would govern for deflection checks, therefore, no need to check serviceability here.

1.2D + 1.6L Mu and Vu diagrams





Note: We only need to consider **three segments (30 to 40, 40 to 50 and 50 to 60 ft)** for the girder here. The Lb for moments is **10 ft** for this case. Each segment has the unique C_b, and M_{u_max}.
Recall: RISA uses an opposite moment convention to ours, so what it shows as "positive" has tension on the top and compression on the bottom of the girder.

Section 1-1 from x = 30 to 40 ft

$x := 32.5\ ft \qquad M_A := -460.4\ kip \cdot ft \qquad x := 35\ ft \qquad M_B := -240.8\ kip \cdot ft$

$x := 37.5\ ft \qquad M_C := -21.5\ kip \cdot ft \qquad x := 30\ ft \qquad M_{max} := -679\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 2.0669$$

Compute M_{u_eff} for section 1 as shown above

Maximum factored moment in (section 1) $\qquad M_{u1} := \text{abs}\left(M_{max}\right) = 679\ kip \cdot ft$

Effective factored moment M_{u_eff} $\qquad M_{u\_eff1} := \frac{M_{u1}}{C_b} = 328.512\ kip \cdot ft$

Section 2-2 from x = 40 to 50 ft

$x := 42.5\ ft \qquad M_A := 224.5\ kip \cdot ft \qquad x := 45\ ft \qquad M_B := 251.2\ kip \cdot ft$

$x := 47.5\ ft \qquad M_C := 277.5\ kip \cdot ft \qquad x := 50\ ft \qquad M_{max} := 303.5\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 1.1603$$

Compute M_{u_eff} for section 2 as shown above

Maximum factored moment in (section 2) $\qquad M_{u2} := \text{abs}\left(M_{max}\right) = 303.5\ kip \cdot ft$

Effective factored moment M_{u_eff} $\qquad M_{u\_eff2} := \frac{M_{u2}}{C_b} = 261.564\ kip \cdot ft$

Section 3-3 from x = 50 to 60 ft

$x := 52.5\ ft$ $M_A := 137.5\ kip \cdot ft$ $x := 55\ ft$ $M_B := -28.8\ kip \cdot ft$

$x := 57.5\ ft$ $M_C := -195.5\ kip \cdot ft$ $x := 60\ ft$ $M_{max} := -362.5\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 2.2427$$

Compute M_{u_eff} for section 3 as shown above

Maximum factored moment in Section 3 $M_{u3} := \text{abs}\left(M_{max}\right) = 362.5\ kip \cdot ft$

Effective factored moment M_{u_eff} $M_{u\_eff3} := \dfrac{M_{u3}}{C_b} = 161.636\ kip \cdot ft$

Factored shear V_u (diagram) $V_u := 100.1\ kip$

Use Table 3-10, select the lightest section regarding to L_b and M_{u_eff}

Unbraced length for both **Sections** $L_b := 10\ ft$

Max factored moment M_{u_eff} $M_u := \max\left(M_{u1}, M_{u2}, M_{u3}\right) = 679\ kip \cdot ft$

Effective factored moment M_{u_eff} $M_{u\_eff} := \max\left(M_{u\_eff1}, M_{u\_eff2}, M_{u\_eff3}\right) = 328.5\ kip \cdot ft$

Note: Find the right and upper bolded (lightest in group) line as the order below with L_b = 10 ft and M_{u_eff} = 328.5 kip*ft.

General process shown as below table is:
check **Table 3-10** to ensure $\phi_b M_n > M_{u\_eff}$
check **Table 3-2** to ensure $\phi_b M_{px} > M_u$
if both of those are satisfied, the shape will work for strength.
If serviceability is also considered (as in this assignment), then also check **Table 3-3** to ensure $I_{x\_trial} \geq I_{x\_req}$
When all those criteria are satisfied, then proceed to verify the capacity using the Specifications.

Selection process: Firstly, check if Case 2 section **W18X76** works for the Case 3. **It does not work due to** $\phi_b M_{px}$ is smaller than ***Mu=679 kip*ft*****. Then, we need** using **Table 3-10** and find the first try section **W21X48** on **Pg. 3-117**, but the $\phi_b M_{px}$ is smaller than *Mu* (**not good**). Then find sections on **Pg. 3-117.** You can find the $\phi_b M_{px}$ **of W18X55, W21X55, W21X62, W16X67, W21X68, and W24X68** on **Pg. 3-117 is also** smaller than required ***Mu***. Then you may move to **Pg. 3- 115** and try the **W24X76** is a good trial section as shown in below table, because its $\phi_b M_{px}$ is larger than *and the moment of inertia is greater than* ***Ix, required*** *according to the load case 1.*
Therefore, try the **W24X76** for the load case 3.

Trial section	Mu(kip-ft)	φbMpx(kip-ft)	φ Mn(LTB)	φ Mn(min)	Ix,min(in^4)	Ix(in^4)	
W18X76	679	611			1138.4		No
W21X48	679	398			1138.4		No
W18X55	679	420			1138.4		No
W21X55	679	473			1138.4		No
W21X62	679	540			1138.4		No
W16X67	679	488			1138.4		No
W21X68	679	600			1138.4		No
W24X68	679	664			1138.4		No
W24X76	679	750	? > 679		1138.4	2100	Worth to try

Analyze the trial section to verify capacity

Check if $\phi_b M_n \geq M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 1.92\ in$

Yield stress $F_y := 50\ ksi$

L_p Pg. 16-48. (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 6.782\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 2.68\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 2.33\ in$

Elastic section modulus Table 1-1 $S_x := 176\ in^3$

Distance between flanges centroids Table 1-1 $h_o := 23.2\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 19.496\ ft$$

$check := \textbf{if}\left(L_b \le L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \textbf{if}\left(L_p \le L_b \le L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Inelastic LTB”}$

$check := \textbf{if}\left(L_r \le L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Not Elastic LTB”}$

Note: Therefore, it is in the **Zone 2** range (**inelastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.

$L_p < L_b \le L_r$ → **Zone 2** Inelastic LTB → $\phi_b M_n = \min \left\{ \begin{array}{c} C_b\left[\phi_b M_{px} - \phi_b BF\left(L_b - L_p\right)\right] \\ \phi_b M_{px} \end{array} \right\}$

Modulus of elasticity E $\quad E := 29000\ \textit{ksi}$

Plastic section modulus (Table 1-1) $\quad Z_x := 200\ \textit{in}^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\quad \lambda_f := 6.61$

λ_p ratio (Table B4.1b case 10) $\quad \lambda_p := 0.38 \cdot \sqrt{\dfrac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\quad \lambda_r := 1.0 \cdot \sqrt{\dfrac{E}{F_y}} = 24.083$

$check := \textbf{if}\left(\lambda_f \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\quad \lambda_w := 49$

λ_r ratio (Table B4.1 b case 15) $\quad \lambda_p := 3.76 \cdot \sqrt{\dfrac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\quad \lambda_r := 5.70 \cdot \sqrt{\dfrac{E}{F_y}} = 137.274$

$check := \textbf{if}\left(\lambda_w \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 833.33\ \mathit{kip} \cdot \mathit{ft}$

Factored plastic moment (confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 750\ \mathit{kip} \cdot \mathit{ft}$

BF term shown in F2-2 $BF := \dfrac{\left(M_p - 0.7 \cdot F_y \cdot S_x\right)}{L_r - L_p} = 25.17\ \mathit{kip}$

Factored moment capacity (F2-2) $\phi_b M_n := min\left(C_b \cdot \left(\phi_b \cdot M_p - \phi_b \cdot BF \cdot \left(L_b - L_p\right)\right), \phi_b \cdot M_p\right) = 750\ \mathit{kip} \cdot \mathit{ft}$

$check := \textbf{if}\left(\phi_b M_n \ge M_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Note: We only need check segment 1 for the moment capacity here because the segment has the smallest Lb and largest Mu among the three sections.

Check if $I_{x\_trial} \ge I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 2100\ \mathit{in}^4$

$check := \textbf{if}\left(I_{x\_trial} \ge I_{x\_req}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check if $\phi_v V_{nx} \ge V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 49$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$check := \textbf{if}\left(\lambda_w \le \lambda, \text{“YES”}, \text{“NO”}\right) = \text{“YES”}$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 23.9\ \mathit{in}$

Web thickness of section (Table 1-1) $t_w := 0.44\ \mathit{in}$

Area of web (G2-1) $A_w := d \cdot t_w = 10.516\ \mathit{in}^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 315\ \mathit{kip}$

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 315\ \mathit{kip}$

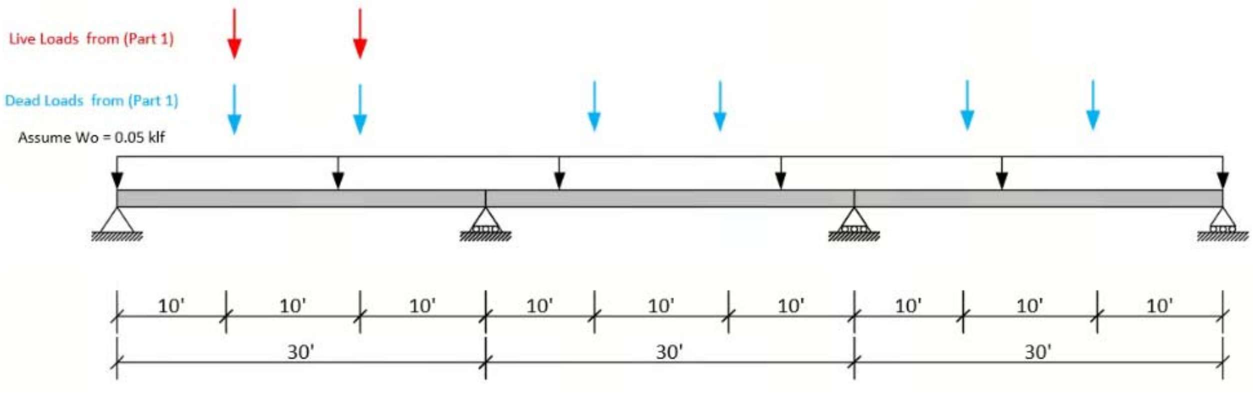
Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 315\ kip$

$$check := \textbf{if}\left(\phi_v V_{nx} \geq V_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Note: We assumed the girder weight is 50 lb/ft to determine the M_u **&** V_u**.** But we actually use **W24X76** for the final design which is **heavier** than we assumed. We use **W24X76** which has **750 kip*ft** moment capacity and **315 kip** shear capacity. Therefore, the moment, shear and moment of inertia **would not be a problem for this little self-weight change**.

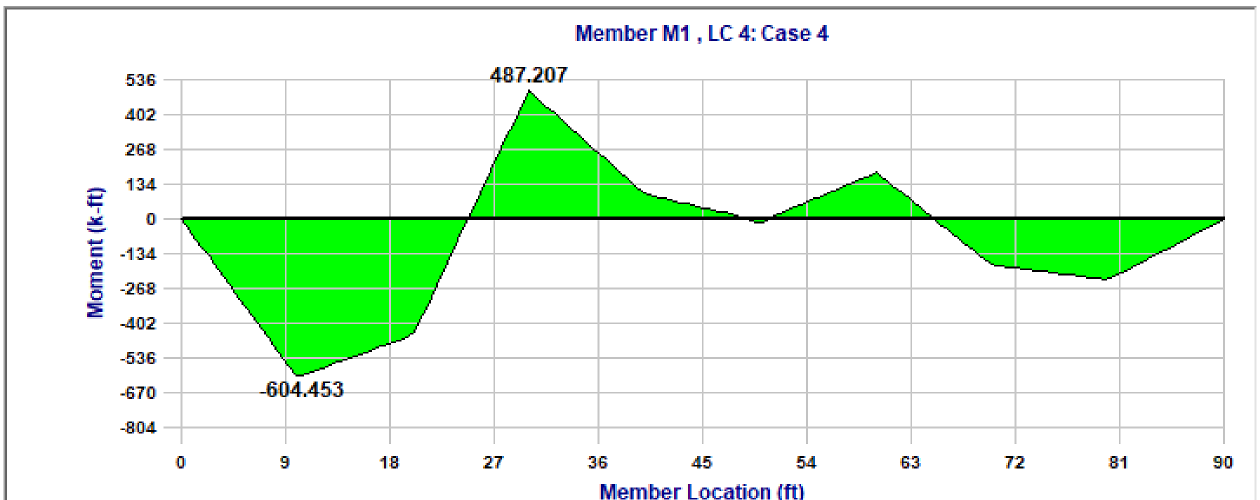
Therefore, using the **W24X76** for case 3.

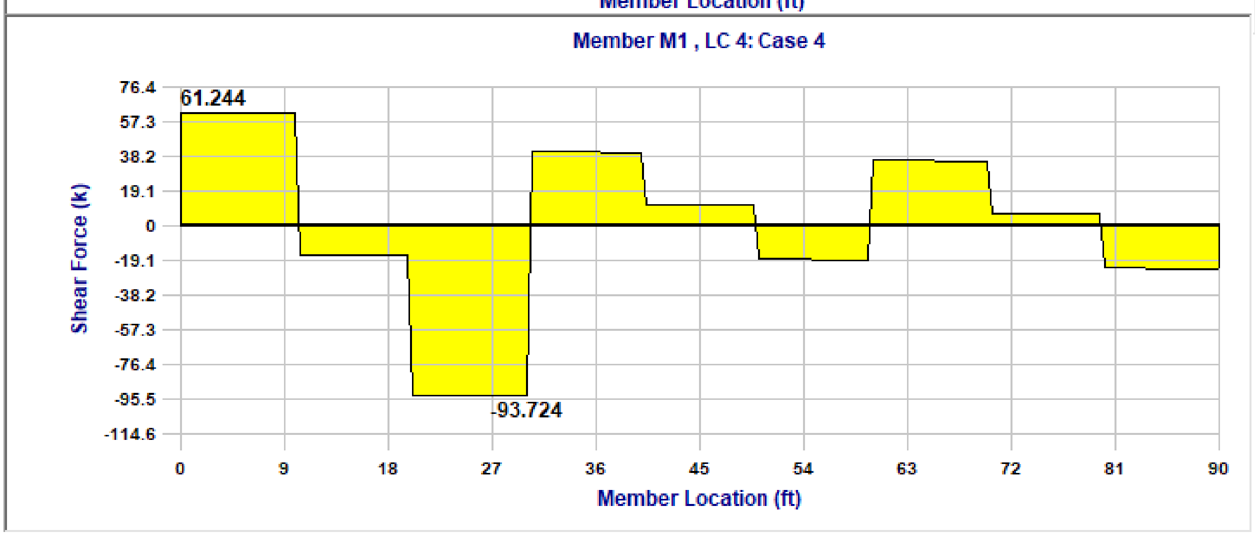
Case (4) live load only on one adjacent bay



Note: We need to use the *Mu* diagram to determine the *Cb* for each load case. RISA 2D was used to get the moment and shear diagram for the **1.2D + 1.6L load combination.** Recall that the Case 1 would govern for deflection checks, therefore, no need to check serviceability here.

1.2D + 1.6L Mu and Vu diagrams





Note: We only need to consider **two segments (30 to 50 and 50 to 60 ft)** for the girder here. The L_b for moments is either **20 ft or 10 ft** for this case. Each segment has the unique L_b C_b, and M_{u_max}.

Section 1-1 from x = 30 to 50 ft

$x := 35\ ft$ $M_A := -289\ kip \cdot ft$ $x := 40\ ft$ $M_B := -90.8\ kip \cdot ft$

$x := 45\ ft$ $M_C := -37.1\ kip \cdot ft$ $x := 30\ ft$ $M_{max} := -488\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 2.3814$$

Compute M_{u_eff} for section 1 as shown above

Maximum factored moment in (section 1) $M_{u1} := \text{abs}\left(M_{max}\right) = 488\ kip \cdot ft$

Effective factored moment M_{u_eff} $M_{u\_eff1} := \frac{M_{u1}}{C_b} = 204.92\ kip \cdot ft$

Section 2-2 from x = 50 to 60 ft

$x := 52.5\ ft$ $M_A := -30.7\ kip \cdot ft$ $x := 55\ ft$ $M_B := -77\ kip \cdot ft$

$x := 57.5\ ft$ $M_C := -123.7\ kip \cdot ft$ $x := 60\ ft$ $M_{max} := -170.7\ kip \cdot ft$

$$C_b := \frac{12.5 \cdot \text{abs}\left(M_{max}\right)}{2.5 \cdot \text{abs}\left(M_{max}\right) + 3 \cdot \text{abs}\left(M_A\right) + 4 \cdot \text{abs}\left(M_B\right) + 3 \cdot \text{abs}\left(M_C\right)} = 1.7812$$

Compute M_{u_eff} for section 2 as shown above

Maximum factored moment in (section 2) $M_{u2} := \text{abs}\left(M_{max}\right) = 170.7\ kip \cdot ft$

Effective factored moment M_{u_eff} $M_{u\_eff2} := \frac{M_{u2}}{C_b} = 95.836\ kip \cdot ft$

Factored shear V_u (diagram) $V_u := 93.7\ kip$

Use Table 3-10, select the lightest section regarding to L_b *and* M_{u_eff}

Unbraced length for **Section 1** $L_b := 20\ ft$

Max factored moment M_{u_eff} $M_u := \max(M_{u1}, M_{u2}) = 488\ kip \cdot ft$

Effective factored moment M_{u_eff} $M_{u\_eff} := \max(M_{u\_eff1}, M_{u\_eff2}) = 204.9\ kip \cdot ft$

Selection process: Based on the above calculations, it seems that the Case 4 is not a governing case comparing to Case 2 or Case 3. Because it has the smaller Lb than Case 2 and smaller Mu than Case 3.

We may can try to come up with the design section works for Case 2 and 3 and then check if it works for Case 4.

Find the design to accommodate the requirements for Case 2 and 3

Summary parameters for Case 2

Unbraced length $L_b := 30\ ft$

Max factored moment M_{u_eff} $M_u := 425\ kip \cdot ft$

$C_b := 1.8765$

Effective factored moment M_{u_eff} $M_{u\_eff} := \dfrac{M_u}{C_b} = 226.485\ kip \cdot ft$

Summary parameters for Case 3

Unbraced length $L_b := 10\ ft$

Max factored moment M_{u_eff} $M_u := 679\ kip \cdot ft$

$C_b := 2.0669$

Effective factored moment M_{u_eff} $M_{u\_eff} := \dfrac{M_u}{C_b} = 328.511\ kip \cdot ft$

Note: These are two interesting cases. There is a larger *Lb* but a smaller *Mu* for Case 2. For case 3, there is a smaller *Lb* but higher *Mu*. It just needs to be safe in each of the 4 cases. Looking at your. A good way you can do it is to use the spreadsheet and try to select a lightest section both good for Case 2 and Case 3 and then check if it works for case 1 & 4.

Selection process: Firstly, we can check if **W24X76** used for Case 3 works for Case 2**.** If it is not, you may use the spreadsheet to get the reasonable efficient section. The spreadsheet will be uploaded to Canvas for this problem.

For Case 2 (W24X76)

Analyze the trial section to verify capacity

Unbraced length $L_b := 30\ ft$

Max factored moment M_u $M_u := 425\ kip \cdot ft$

Max factored shear V_u $V_u := 91.65\ kip$

Check if $\phi_b M_n \geq M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 1.92\ in$

Yield stress $F_y := 50\ ksi$

L_p (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 6.782\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 2.68\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 2.33\ in$

Elastic section modulus Table 1-1 $S_x := 176\ in^3$

Distance between flanges centroids Table 1-1 $h_o := 23.2\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

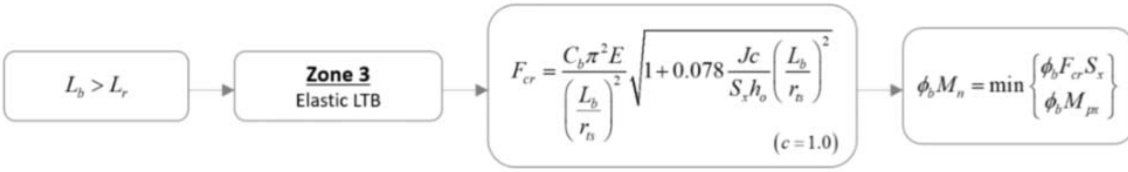
$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 19.496\ ft$$

$check := \textbf{if}\left(L_b \leq L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \textbf{if}\left(L_p \leq L_b \leq L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Not Inelastic LTB”}$

$check := \textbf{if}\left(L_r \leq L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Elastic LTB”}$

Note: Therefore, it is in the **Zone 3** range (**elastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Modulus of elasticity E $E := 29000\ ksi$

Plastic section modulus (Table 1-1) $Z_x := 200\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 6.61$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$check := \textbf{if}\left(\lambda_f \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 49$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$check := \textbf{if}\left(\lambda_w \le \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$

Note: Both flange and web elements are compact, therefore using Equation F2-4 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 833.33\ kip \cdot ft$

Factored plastic moment (confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 750\ \boldsymbol{kip \cdot ft}$

BF term shown in F2-4 $F_{cr} := \dfrac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \dfrac{(J \cdot c)}{S_x \cdot h_o} \cdot \left(\dfrac{L_b}{r_{ts}}\right)^2} = 36.94\ \boldsymbol{ksi}$

Factored moment capacity (F2-3) $\phi_b M_n := min\left(\phi_b \cdot F_{cr} \cdot S_x , \phi_b \cdot M_p\right) = 487.623\ \boldsymbol{kip \cdot ft}$

$check := \textbf{if}\left(\phi_b M_n \ge M_u , \text{“OK”} , \text{“NG”}\right) = \text{“OK”}$

Check if $I_{x\_trial} \ge I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 2100\ \boldsymbol{in}^4$

$check := \textbf{if}\left(I_{x\_trial} \ge I_{x\_req} , \text{“OK”} , \text{“NG”}\right) = \text{“OK”}$

Check if $\phi_v V_{nx} \ge V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 49$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$check := \textbf{if}\left(\lambda_w \le \lambda , \text{“YES”} , \text{“NO”}\right) = \text{“YES”}$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 23.9\ \boldsymbol{in}$

Web thickness of section (Table 1-1) $t_w := 0.44\ \boldsymbol{in}$

Area of web (G2-1) $A_w := d \cdot t_w = 10.516\ \boldsymbol{in}^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 315.5\ \boldsymbol{kip}$

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 315.5\ \boldsymbol{kip}$

Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 315.5\ \boldsymbol{kip}$

$check := \textbf{if}\left(\phi_v V_{nx} \ge V_u , \text{“OK”} , \text{“NG”}\right) = \text{“OK”}$

For Case 3 (W24X76)

Analyze the trial section to verify capacity

Unbraced length $L_b := 10\ ft$

Max factored moment M_u $M_u := 679\ kip \cdot ft$

Max factored shear V_u $V_u := 100.1\ kip$

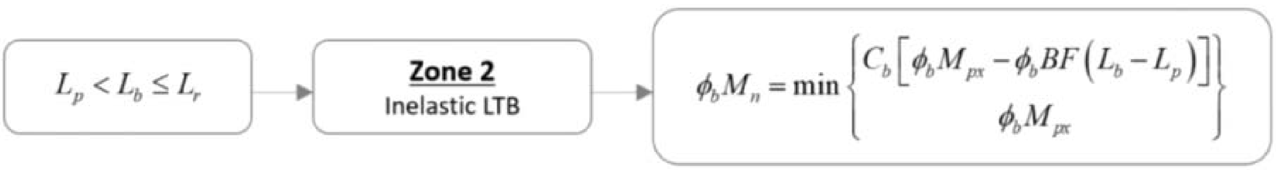
Check if $\phi_b M_n \geq M_u$

$check := \text{if}\left(L_b \leq L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \text{if}\left(L_p \leq L_b \leq L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Inelastic LTB”}$

$check := \text{if}\left(L_r \leq L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Not Elastic LTB”}$

Note: Therefore, it is in the **Zone 2** range (**inelastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Plastic section modulus (Table 1-1) $Z_x := 200\ in^3$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 833.33\ kip \cdot ft$

Factored plastic moment
(confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 750\ kip \cdot ft$

BF term shown in F2-2 $BF := \dfrac{\left(M_p - 0.7 \cdot F_y \cdot S_x\right)}{L_r - L_p} = 25.17\ kip$

Factored moment capacity (F2-2) $\phi_b M_n := min\left(C_b \cdot \left(\phi_b \cdot M_p - \phi_b \cdot BF \cdot \left(L_b - L_p\right)\right), \phi_b \cdot M_p\right) = 750\ kip \cdot ft$

$check := \text{if}\left(\phi_b M_n \geq M_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

For Case 1

Analyze the trial section to verify capacity

Unbraced length $L_b := 10\ ft$

Max factored moment M_u $M_u := 425\ kip \cdot ft$

Max factored shear V_u $V_u := 77.5\ kip$

For Case 4

Analyze the trial section to verify capacity

Unbraced length $L_b := 20\ ft$

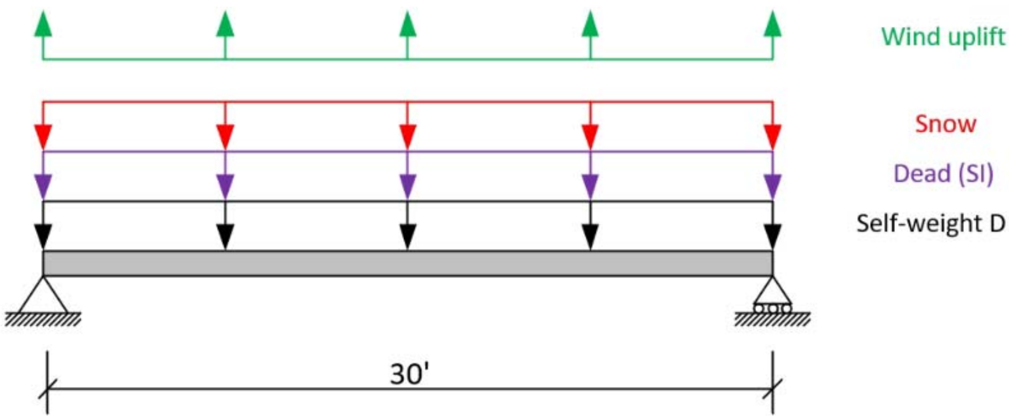
Max factored moment M_u $M_u := 488\ kip \cdot ft$

Max factored shear V_u $V_u := 93.8\ kip$

Note: We can conclude the ***W24X76*** *section would pass for case 1 & 4 because the* $\phi_b M_n$ *= 750 kip*ft*

Summary: Using the ***W24X76*** *for this part 3 preliminary design based on the four different cases. The case 1 would govern for the maximum deflection and the case 3 would govern for the maximum factored moment and shear Mu and Vu.* ***W24X76*** *has been evaluated and passed with respect to the moment, shear and deflection checks.*

Part 4: A simple-span typical *beam* at the high roof (note: this beam takes the place of a typical roof joist in the as-built drawings), in the bay bounded by column lines B to C and 30 to 32. Consider all applicable load combinations, including wind uplift. Use a wind uplift pressure of **50 psf.** (*Aside: This increased wind pressure reflects that localized pressures over a small area at a single beam may be higher than the average acting across the entire roof, which was used in Assignment 1.*) Assume **that snow only occurs as a uniform load on the high roof**. (*This is not technically correct, but a simplification for this assignment.*) Note that typical roof *beams* are **continuously laterally** supported by the roof deck in **positive moment**, but **unbraced full length** for **negative moment**. If your design for positive moment requires bracing for negative moment, specify locations where bracing is required, and provide a simple conceptual sketch illustrating the bracing.



The dead and snow loads are come from the Project 1B solution.

Determine the factored moment and shear, M_u & V_u (Positive moment)

Floor beam length $L := 30\ ft$

Tributary width $W := 5\ ft$

Dead load on the roof beam $w_D := 25\ psf \cdot W = 0.125\ klf$

Snow load on the roof beam $w_S := 25\ psf \cdot W = 0.125\ klf$

Assume 25 lb/ft self-weight of the roof beam $w_O := 25\ plf = 0.025\ klf$

Mid-span dead load moment $M_{Mid\_D} := \left(w_D + w_O\right) \cdot \frac{L^2}{8} = 16.875\ kip \cdot ft$

End support dead load shear $V_{End\_D} := \left(w_D + w_O\right) \cdot \frac{L}{2} = 2.25\ kip$

Mid-span snow load moment $M_{Mid\_S} := w_S \cdot \frac{L^2}{8} = 14.063\ kip \cdot ft$

End support snow load shear $V_{End\_S} := w_S \cdot \frac{L}{2} = 1.875\ kip$

Control load combination ***1.2D + 1.6S***

Factored moment at the mid $M_u := 1.2 \cdot M_{Mid\_D} + 1.6 \cdot M_{Mid\_S} = 42.75\ kip \cdot ft$

Factored shear at the end $V_u := 1.2 \cdot V_{End\_D} + 1.6 \cdot V_{End\_S} = 5.7\ kip$

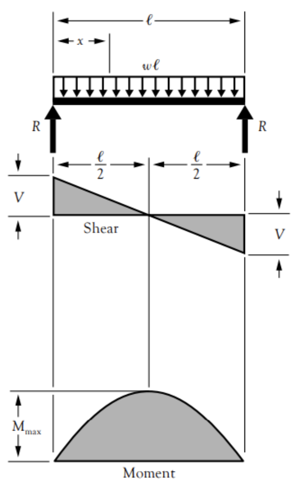
Determine the required moment of inertia, I_x

Modulus of elasticity of steel $E := 29000\ ksi$

Max allowable deflection at the mid for total load $\Delta_{T\_max} := \dfrac{L}{240} = 1.5\ in$

Required moment of inertia of total load $I_{x\_req} := \dfrac{\left(5 \cdot \left(w_D + w_O\right) \cdot L^4 + 5 \cdot w_S \cdot L^4\right)}{384 \cdot E \cdot \left(\Delta_{T\_max}\right)} = 115.216\ in^4$

Figure 1 Simple Beam – Uniformly Distributed Load



$R = V \ldots = \dfrac{w\ell}{2}$

$V_x \ldots = w\left(\dfrac{\ell}{2} - x\right)$

M_{max} (at center) $\ldots = \dfrac{w\ell^2}{8}$

$M_x \ldots = \dfrac{wx}{2}(\ell - x)$

Δ_{max} (at center) $\ldots = \dfrac{5w\ell^4}{384\,EI}$

$\Delta_x \ldots = \dfrac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$

Use Table 3-2, select the lightest section based on $\phi b M_{px}$ ***values and assume*** F_y ***= 50 ksi***

Find the lightest section with $\phi_b M_{px}$ >= M_u **(Table 3-2)**

Try **W10X12** (bolded on page 3-27) with $\phi_b M_{px}$ = 46.9 kip-ft. **Bold** means it is the lightest in that group.

Check if selected trial section has $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 53.8\ in^4$

$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{“OK”}, \text{“NG”}\right) = \text{“NG”}$

Then, refer to Table 3-3 and select the most economical section based on I_x.Therefore, try **W12X19** (bolded on page 3-29).

Analyze the trial section to verify capacity

Check if $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 130 \ in^4$

$$check := \text{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Check if $\phi_b M_{px} \geq M_u$

Yield stress (Table 1-12) $F_y := 50 \ ksi$

Modulus of elasticity E $E := 29000 \ ksi$

Plastic section modulus (Table 1-1) $Z_x := 24.7 \ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 5.72$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$$check := \text{if}\left(\lambda_f \leq \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 46.2$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$$check := \text{if}\left(\lambda_w \leq \lambda_p, \text{“C”}, \text{“NC”}\right) = \text{“C”}$$

Note: Both flange and web elements are compact, therefore using Equation F2-1 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_n := F_y \cdot Z_x$

The design flexure strength $\phi_b M_n := \phi_b \cdot M_n = 92.6\ \mathbf{kip} \cdot \mathbf{ft}$

The design flexure strength (Table 3-2) $\phi_b M_{px} := 92.6\ \mathbf{kip} \cdot \mathbf{ft}$

$$check := \text{if}\left(\phi_b M_n \geq M_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Check if $\phi_v V_{nx} \geq V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 46.2$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$$check := \text{if}\left(\lambda_w \leq \lambda, \text{“YES”}, \text{“NO”}\right) = \text{“YES”}$$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 12.2\ \mathbf{in}$

Web thickness of section (Table 1-1) $t_w := 0.235\ \mathbf{in}$

Area of web (G2-1) $A_w := d \cdot t_w = 2.867\ \mathbf{in}^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 86\ \mathbf{kip}$

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 86\ \mathbf{kip}$

Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 86\ \mathbf{kip}$

$$check := \text{if}\left(\phi_v V_{nx} \geq V_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Determine the factored moment and shear, M_u & V_u (Negative moment)

Note: Firstly, verify if W12X19 would meet the requirement of the moment capacity for the negative moment. The result showed that the $\phi_b M_n$=11.76 kip*ft which is smaller than M_u. Therefore, we need to increase the section to **W12X22 for this design.**

Floor beam length $L := 30\ ft$

Tributary width $W := 5\ ft$

Dead load on the roof beam $w_D := 25\ psf \cdot W = 0.125\ klf$

Wind load on the roof beam $w_W := 50\ psf \cdot W = 0.25\ klf$

Try roof beam as **W12X22** $w_O := 22\ plf = 0.022\ klf$

Mid-span dead load moment $M_{Mid\_D} := (w_D + w_O) \cdot \frac{L^2}{8} = 16.538\ kip \cdot ft$

End support dead load shear $V_{End\_D} := (w_D + w_O) \cdot \frac{L}{2} = 2.205\ kip$

Mid-span wind uplift load moment $M_{Mid\_W} := w_W \cdot \frac{L^2}{8} = 28.125\ kip \cdot ft$

End support wind uplift load shear $V_{End\_W} := w_W \cdot \frac{L}{2} = 3.75\ kip$

Control load combination ***0.9D + 1.0W***

Factored moment at the mid $M_u := -0.9 \cdot M_{Mid\_D} + 1.0 \cdot M_{Mid\_W} = 13.241\ kip \cdot ft$

Factored shear at the end $V_u := -0.9 \cdot V_{End\_D} + 1.0 \cdot V_{End\_W} = 1.766\ kip$

Determine the required moment of inertia, I_x

Modulus of elasticity of steel $E := 29000\ ksi$

Max allowable deflection at the mid for total load $\Delta_{T\_max} := \frac{L}{240} = 1.5\ in$

Note: The maximum deflection along the beam is located at the middle of the girder.

Required moment of inertia of total load $I_{x\_req} := \frac{\text{abs}\left(5 \cdot (w_D + w_O - w_W) \cdot L^4\right)}{384 \cdot E \cdot (\Delta_{T\_max})} = 43.153\ in^4$

Use Table 3-10, select the lightest section regarding to L_b ***and*** M_{u_eff}

Unbraced length $L_b := L = 30\ ft$

C_b From Table 3-1 $C_b := 1.14$

Effective factored moment M_{u_eff} $M_{u\_eff} := \dfrac{M_u}{C_b} = 11.615\ kip \cdot ft$

Note: Verify the <u>W12X22</u> section is good for the negative moment consideration

Analyze the trial section to verify capacity

Check if $\phi_b M_n \geq M_u$

Radius of gyration about y- axis (Table 1-1) $r_y := 0.848\ in$

Yield stress $F_y := 50\ ksi$

L_p Pg. 16-48. (confirmed with Table 3-2) $L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 2.995\ ft$

c= 1 for doubly symmetric Pg. 16-48 $c := 1.0$

Torsion constant Table 1-1 $J := 0.293\ in^4$

Radius of gyration of LTB Table 1-1 $r_{ts} := 1.04\ in$

Elastic section modulus Table 1-1 $S_x := 25.4\ in^3$

Distance between flanges centroids Table 1-1 $h_o := 11.9\ in$

L_r Pg. 16-48.
(confirmed with Table 3-2)

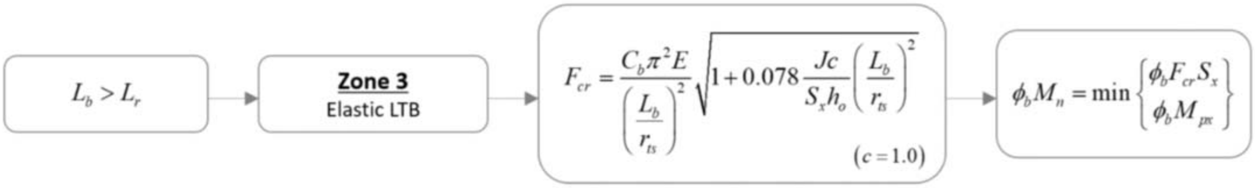
$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 9.133\ ft$$

$check := \text{if}\left(L_b \leq L_p, \text{“Full plastic moment”}, \text{“Not Full plastic”}\right) = \text{“Not Full plastic”}$

$check := \text{if}\left(L_p \leq L_b \leq L_r, \text{“Inelastic LTB”}, \text{“Not Inelastic LTB”}\right) = \text{“Not Inelastic LTB”}$

$check := \text{if}\left(L_r \leq L_b, \text{“Elastic LTB”}, \text{“Not Elastic LTB”}\right) = \text{“Elastic LTB”}$

Note: Therefore, it is in the **Zone 3** range (**elastic LTB**). Then we will use **equations (F-2) shown in figure above** to check **moment** capacity.



Modulus of elasticity E $E := 29000\ ksi$

Plastic section modulus (Table 1-1) $Z_x := 29.3\ in^3$

Flange slenderness check (for selected section)

Width to thickness ratio b/2*tf (λ_f) (Table 1-1) $\lambda_f := 4.74$

λ_p ratio (Table B4.1b case 10) $\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$

λ_r ratio (Table B4.1b case 10) $\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$$check := \text{if}\left(\lambda_f \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$$

Web slenderness check (for selected section)

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 41.8$

λ_r ratio (Table B4.1 b case 15) $\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

λ_r ratio (Table B4.1b case 15) $\lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

$$check := \text{if}\left(\lambda_w \le \lambda_p, \text{"C"}, \text{"NC"}\right) = \text{"C"}$$

Note: Both flange and web elements are compact, therefore using Equation F2-4 to calculate the moment capacity

Phi factor (Section F1) $\phi_b := 0.90$

Plastic moment (Equation F2-1) $M_p := F_y \cdot Z_x = 122.08\ \boldsymbol{kip \cdot ft}$

Factored plastic moment (confirmed with Table 3-2) $\phi_b M_p := \phi_b \cdot M_p = 109.875\ \boldsymbol{kip \cdot ft}$

BF term shown in F2-4 $$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{(J \cdot c)}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 8.64\ \boldsymbol{ksi}$$

Factored moment capacity (F2-3) $\phi_b M_n := min\left(\phi_b \cdot F_{cr} \cdot S_x, \phi_b \cdot M_p\right) = 16.453\ \boldsymbol{kip \cdot ft}$

$check := \textbf{if}\left(\phi_b M_n \geq M_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check if $I_{x\_trial} \geq I_{x\_req}$

Selected section moment of inertial (Table 1-1) $I_{x\_trial} := 156\ \boldsymbol{in}^4$

$check := \textbf{if}\left(I_{x\_trial} \geq I_{x\_req}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check if $\phi_v V_{nx} \geq V_u$

Width to thickness ratio h/tw (λ_w) (Table 1-1) $\lambda_w := 41.8$

determine ϕ_v & C_{v1} according to G2-2 $\lambda := 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$check := \textbf{if}\left(\lambda_w \leq \lambda, \text{“YES”}, \text{“NO”}\right) = \text{“YES”}$

The web shear strength coefficient $C_{v1} := 1.0$

Phi factor for shear (G2-2) $\phi_v := 1.0$

Depth of section (Table 1-1) $d := 12.3\ \boldsymbol{in}$

Web thickness of section (Table 1-1) $t_w := 0.26\ \boldsymbol{in}$

Area of web (G2-1) $A_w := d \cdot t_w = 3.198\ \boldsymbol{in}^2$

The nominal shear strength (G2-1) $V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 95.9$ ***kip***

The design shear strength $\phi_v V_n := \phi_v \cdot V_n = 95.9$ ***kip***

Selected section factored shear strength (Table 3-2) $\phi_v V_{nx} := 95.9$ ***kip***

$check := \textbf{if}\left(\phi_v V_{nx} \geq V_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

*Summary: Using the **W12X22** beam for this part 4 preliminary design. Using F-2 for the flexure strength calculation because **W12X22** has **compact** web and flanges according to Table 4-1.b **case 10 &15**. Be careful with the ϕ**=0.9**. Using the G1 & G2-1a for the shear strength calculation. Be careful with the ϕ**=1.0**. The validation of factored moment and shear strength using Table 3-2 in AISC is provided above.*