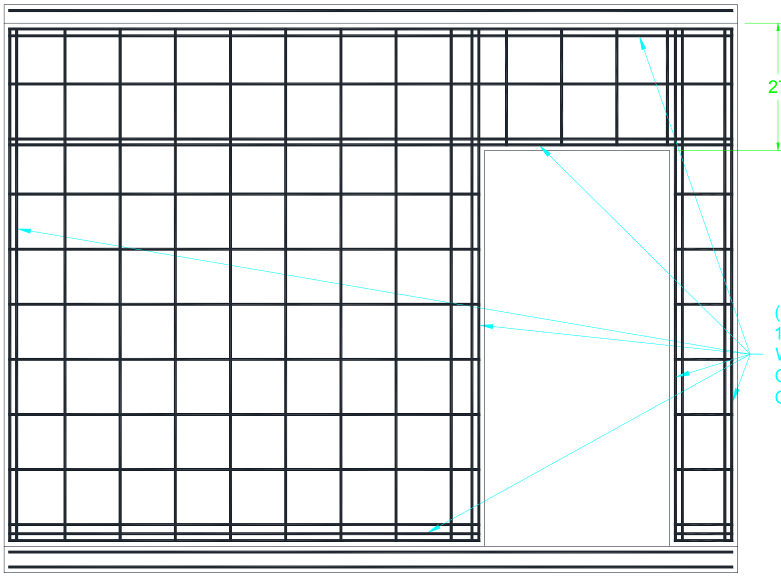
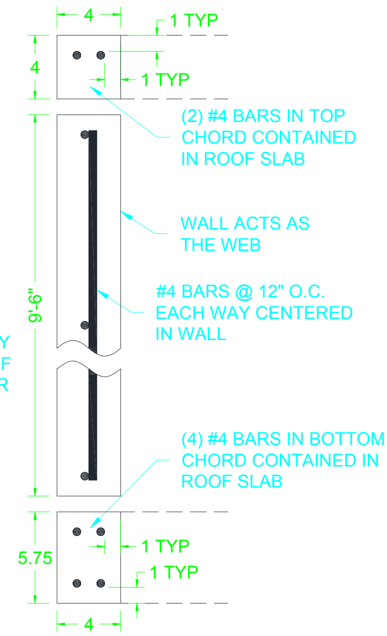
DEEP BEAM WALL TO SPAN PIERS

COMPOSITE BEAM: ROOF CHORD + WALL + FLOOR CHORD





$L := L_{act} = 14\,ft$ Length of the member

$b_w := 4 \cdot in$ Width of concrete

$\rho_{conc} := 115 \cdot pcf$ Density of concrete

$d_b := 0.5 \cdot in$ Diameter of the reinforcement

$c_c := 1 \cdot in$ Concrete cover from concrete surface to the surface of the tension reinforcement

$t := H_{wall} + t_{r\_e_1} + t_{f\_act} = 123.75\,in$ Thickness of the beam

$d := t - (c_c + 0.5 \cdot d_b) = 122.5 \cdot in$ Distance from extreme compression fiber to centroid of tension reinforcement

$d := 27 \cdot in$ **Conservatively manually set the variable to the depth of wall above the door. Most locations would be the full value calculated above, but this shall be used for ease of calculation.**

Unfactored Design Loads

Distributed loads of the roof are applied as distributed loads on the composite beam because the roof bears continuously on the wall:

$$w_{DL\_r} := \frac{W_r(B_{act}, L_{act}, \rho_{lw})}{2 \cdot L_{act}} \qquad w_{DL\_r} = 253 \cdot plf$$

$$w_{LL\_r} := LL_r \cdot (0.5 \cdot B_{act}) \qquad w_{LL\_r} = 750 \cdot plf$$

$$w_{DL\_w} := \frac{W_{w\_l}(B_{act}, L_{act}, \rho_{lw})}{2 \cdot L_{act}} \qquad w_{DL\_w} = 497 \cdot plf$$

Distributed loads of floor are applied as point loads on the composite beam at the embed plates:

$$w_{DL\_f} := \frac{W_f(B_{act}, L_{act}, \rho_{f\_act})}{2 \cdot L_{act}} \qquad w_{DL\_f} = 298 \cdot plf$$

$$w_{LL} := LL_f \cdot (0.5 \cdot B_{act}) \qquad w_{LL} = 1000 \cdot plf$$

Floor Embed Locations:

$d_{e1} := 7.75 \cdot in$

$d_{e2} := 3 \cdot ft + 7.75 \cdot in$

$d_{e3} := 6 \cdot ft + 7.75 \cdot in$

$d_{e4} := L - 9.75 \cdot in - 50.25 \cdot in$

$d_{e5} := L - 9.75 \cdot in$

Member Analysis - LC 1 [1.2DL+1.6LL+0.5Lr]

Support Locations

$x_{r1} := 0$ Support location 1

$x_{r2} := 0.5 \cdot L$ Support location 2

$x_{r3} := L$ Support location 3

Distributed Loads

	Start	End
$w_1 := 1.2 \cdot (w_{DL\_r} + w_{DL\_w}) + 1.6 \cdot w_{LL\_r}$	$x_{w1\_s} := 0$	$x_{w1\_e} := L$

Point Loads

	Start
$P_1 := [1.2 \cdot (w_{DL\_f}) + 0.5 \cdot (w_{LL})] \cdot [d_{e1} + 0.5 \cdot (d_{e2} - d_{e1})]$	$x_{p1} := d_{e1}$
$P_2 := [1.2 \cdot (w_{DL\_f}) + 0.5 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e3} - d_{e1})]$	$x_{p2} := d_{e2}$
$P_3 := [1.2 \cdot (w_{DL\_f}) + 0.5 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e4} - d_{e2})]$	$x_{p3} := d_{e3}$
$P_4 := [1.2 \cdot (w_{DL\_f}) + 0.5 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e5} - d_{e3})]$	$x_{p4} := d_{e4}$
$P_5 := [1.2 \cdot (w_{DL\_f}) + 0.5 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e5} - d_{e4}) + (L - d_{e5})]$	$x_{p5} := d_{e5}$

▶ Shear, Bending, and Deflection Analysis

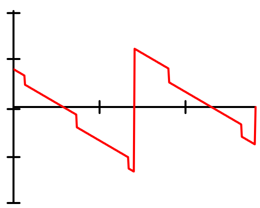
$V_{max_{sub}} = 13.445 \cdot kip$

$M_{max_{sub}} = 209 \cdot kip \cdot in$

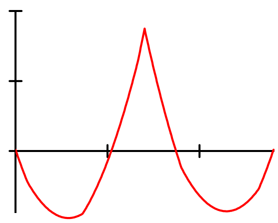
$\delta_{max_{sub}} = 3.468 \times 10^{-3} \cdot in$

$L/\delta_{min_{sub}} = 24224$ ≥ 240

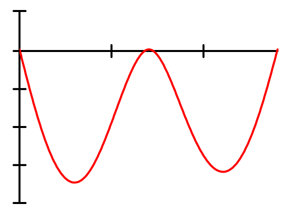
Shear Diagram



Moment Diagram



Deflection Diagram



$R_{r1_{sub}} = 7.87 \cdot kip$ $R_{r2_{sub}} = 25.67 \cdot kip$ $R_{r3_{sub}} = 7.86 \cdot kip$

Member Analysis - LC 2 [1.2DL+1.6Lr+0.5LL]

Support Locations

$x_{r1} := 0$ Support location 1

$x_{r2} := 0.5 \cdot L$ Support location 2

$x_{r3} := L$ Support location 3

Distributed Loads

	Start	End
$w_1 := 1.2 \cdot (w_{DL\_r} + w_{DL\_w}) + 0.5 \cdot w_{LL\_r}$	$x_{w1\_s} := 0$	$x_{w1\_e} := L$

Point Loads

	Start
$P_1 := [1.2 \cdot (w_{DL\_f}) + 1.6 \cdot (w_{LL})] \cdot [d_{e1} + 0.5 \cdot (d_{e2} - d_{e1})]$	$x_{p1} := d_{e1}$
$P_2 := [1.2 \cdot (w_{DL\_f}) + 1.6 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e3} - d_{e1})]$	$x_{p2} := d_{e2}$
$P_3 := [1.2 \cdot (w_{DL\_f}) + 1.6 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e4} - d_{e2})]$	$x_{p3} := d_{e3}$
$P_4 := [1.2 \cdot (w_{DL\_f}) + 1.6 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e5} - d_{e3})]$	$x_{p4} := d_{e4}$
$P_5 := [1.2 \cdot (w_{DL\_f}) + 1.6 \cdot (w_{LL})] \cdot [0.5 \cdot (d_{e5} - d_{e4}) + (L - d_{e5})]$	$x_{p5} := d_{e5}$

$T_{embed} := \max(P_1, P_2, P_3, P_4, P_5)$ $T_{embed} = 6.403 \, kip$ Maximum tension on the embed plate (to be used later in weld analysis for checking embed connection with congruent composite shears)

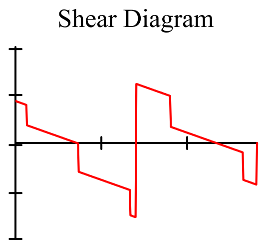
Shear, Bending, and Deflection Analysis

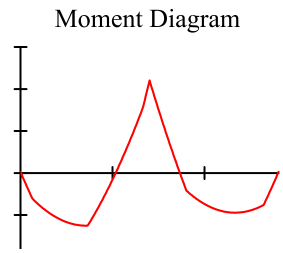
$V_{max_{sub}} = 15.445 \cdot kip$

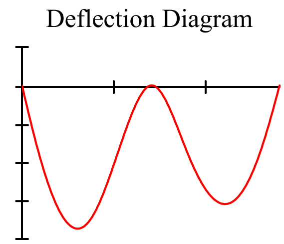
$M_{max_{sub}} = 220 \cdot kip \cdot in$

$\delta_{max_{sub}} = 3.73 \times 10^{-3} \cdot in$

$L/\delta_{min_{sub}} = 22522$ ≥ 240







$R_{r1_{sub}} = 8.74 \cdot kip \quad R_{r2_{sub}} = 27.81 \cdot kip \quad R_{r3_{sub}} = 8.71 \cdot kip$

Check Area of Nonprestressed Tension Reinforcement (22.2, 21.2, 9.6)

$\varepsilon_c := 0.003$ Maximum compression fiber strain (22.2.2.1)

$\varepsilon_{t\_min} := 0.004$ Minimum tension strain (9.3.3.1)

$A_s := 2 \cdot \frac{\pi \cdot d_b^{\,2}}{4}$ $\boxed{A_s = 0.393 \cdot in^2}$ Area of the bar in the transverse direction (strength-axis)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000 \cdot psi \\ 0.65 & \text{if } f'_c > 8000 \cdot psi \\ 0.85 - 0.05 \cdot \left(\frac{f'_c}{ksi} - 4 \right) & \text{otherwise} \end{cases}$$

$\beta_1 = 0.8$ Table 22.2.2.4.3

$A_{min} := \max\left(\frac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d, \frac{200 \cdot psi}{f_y} \cdot b_w \cdot d \right)$ $A_{min} = 0.382 \cdot in^2$ Min flexural reinforcement area (9.6.1.2)

$A_{max} := \frac{0.85 \cdot f'_c \cdot b_w \cdot \beta_1}{f_y} \cdot d \cdot \left(\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_{t\_min}} \right)$ $A_{max} = 2.623 \cdot in^2$ Max flexural steel limit for tension strain to be at least $\varepsilon_{t\_min} = 0.004$.

Check Distribution of Flexural Reinforcement (24.3)

$f_s := \frac{2}{3} \cdot f_y$ Stress in deformed reinforcement (24.3.2.1)

$s := \text{Floor}\left[\min\left[(15 \cdot in) \cdot \left(\frac{40000}{\frac{f_s}{psi}} \right) - 2.5 \cdot c_c, (12 \cdot in) \cdot \left(\frac{40000}{\frac{f_s}{psi}} \right) \right], 0.25 \cdot in \right]$ $s = 12 \cdot in$ Maximum o.c. spacing of flexural tension reinforcement nearest to the extreme tension face (Table 24.3.2)

Check Flexure (21.2, 22.2, 22.3)

$$\varepsilon_t := \frac{0.85 \cdot f'_c \cdot b_w \cdot \beta_1 \cdot d \cdot \varepsilon_c}{\min(A_s, A_{max}) \cdot f_y} - \varepsilon_c$$

$$\phi_b := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.90 & \text{if } \varepsilon_t \ge 0.005 \\ 0.65 + 0.25 \cdot \left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} \right) & \text{otherwise} \end{cases}$$

$\phi_b = 0.9$ Table 21.2.2

$M_u := \max(M_{max})$ $\boxed{M_u = 219.6 \cdot kip \cdot in}$ **Factored moment in beam**

$M_n := \min(A_s, A_{max}) \cdot f_y \cdot \left[d - \frac{\min(A_s, A_{max}) \cdot f_y}{2 \cdot (0.85 \cdot f'_c \cdot b_w)} \right]$ $M_n = 619.84 \cdot kip \cdot in$ Nominal flexural strength of beam (equivalent rectangular concrete stress distribution)

$M_{design} := \phi_b \cdot M_n$ $\boxed{M_{design} = 558 \cdot kip \cdot in}$ **Design bending strength of beam**

A secondary analysis shows that a t_{f_act} x 4" beam with the same steel and cover has a design bending strength of 79*kip-in (not shown for sake of calculation length). This beam, contained within the floor, must support its floor loads between the embedded plates next to the door. Sum the percent utilization of the beam between the door with the percent utilization of the composite beam (remember 'd' was manually set to 27" for extra conservancy). Also realize that the compounding moments are the maximum values for each case. The actual maximum composite beam moment occurs outside of the door location, **AND** the tension zone at the maximum moment is in the roof because of the reverse bending.

$M_{slab} := \frac{(1.6 \cdot w_{LL}) \cdot (d_{e5} - d_{e4})^2}{8} = 42.1 \cdot kip \cdot in$ $\boxed{\frac{M_u}{M_{design}} + \frac{M_{slab}}{79 \cdot (kip \cdot in)} = 0.926}$

Check Shear (21.2, 22.5)

$\boxed{\phi_v := 0.75}$ Table 21.2.1 (21.2.4.1 does not apply because shear is determined by gravity loads)

$\boxed{d_{bv} := 0.5 \cdot in}$ Diameter of the shear reinforcement (vertical bars in the beam)

$\boxed{s_v := 12 \cdot in}$ Spacing of the shear reinforcement

$A_v := \frac{\pi \cdot (d_{bv})^2}{4}$ Area of shear reinforcement per spacing s_v (factor of 1 if single bar, factor of 2 if single hoop, etc.)

$V_u := \max(V_{max})$ $\boxed{V_u = 15.44 \cdot kip}$ **Factored shear in beam**

$V_c := 2 \cdot \lambda(\rho_{conc}) \cdot \sqrt{f'_c \cdot psi} \cdot b_w \cdot d$ $V_c = 12.98 \cdot kip$ (22.5.5.1)

$V_s := \min\left(\frac{A_v \cdot f_y \cdot d}{s_v}, 8 \cdot \lambda(\rho_{conc}) \cdot \sqrt{f'_c \cdot psi} \cdot b_w \cdot d \right)$ $V_s = 26.51 \cdot kip$ (22.5.1.2, 22.5.10.5.3)

$V_{design} := \phi_v \cdot (V_c + V_s)$ $\boxed{V_{design} = 29.6 \cdot kip}$ **Design shear strength of beam**

Calculate Cracked Moment (9.3.2, 24.2)

$y_t := 0.5 \cdot t$ Distance from centroidal axis of gross section, neglecting reinforcement, to tension face

$f_r := \lambda(\rho_{conc}) \cdot 7.5 \cdot \sqrt{f'_c \cdot psi}$ $f_r = 0.451 \cdot ksi$ Modulus of rupture of concrete (19.2.3.1)

$I_{gross} := \frac{b_w \cdot t^3}{12}$ $I_{gross} = 631705 \cdot in^4$ Moment of inertia of gross section about centroidal axis, neglecting reinforcement

Using only thte "chords" of the composite beam:

$A_{tc} := t_{r\_e_1} \cdot b_w$ Area of the top chord

$A_{bc} := t_{f\_act} \cdot b_w$ Area of the bottom chord

$y := \frac{A_{tc} \cdot (t - 0.5 \cdot t_{r\_e_1}) + A_{bc} \cdot (0.5 \cdot t_{f\_act})}{A_{tc} + A_{bc}}$ $y = 4.304\ ft$ Distance from base to centroid

$I_g := A_{bc} \cdot (y - 0.5 \cdot t_{f\_act})^2 + A_{tc} \cdot (t - 0.5 \cdot t_{r\_e_1} - y)^2$ $I_g = 133341 \cdot in^4$

$M_{cr} := \frac{f_r \cdot I_g}{\max(y, t - y)}$ $\boxed{M_{cr} = 834 \cdot kip \cdot in}$ Cracking moment (24.2.3.5) vs $\boxed{M_u = 220 \cdot kip \cdot in}$

Calculate Cracked Moment of Inertia

Again, 'd' was manually set to a conservative value (27") to account for the door opening in the wall.

$E_c := \left(\frac{\rho_{conc}}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{f'_c \cdot psi}$ $E_c = 2878 \cdot ksi$ Modulus of elasticity of concrete (19.2.2.1)

$n := \frac{E_s}{E_c}$ $n = 10.077$ Ratio of modulus of elasticity

$B := \frac{b_w}{n \cdot A_s}$ $B = 1.011 \cdot in^{-1}$ Transformed width of beam assuming steel modulus of elasticity

$kd := \frac{\sqrt{2 \cdot d \cdot B + 1} - 1}{B}$ $kd = 6.387 \cdot in$

$I_{cr} := \frac{b_w \cdot kd^3}{3} + n \cdot A_s \cdot (d - kd)^2$ $I_{cr} = 2029 \cdot in^4$ Cracked moment of inertia (no compression steel)

Calculate Deflection (24.2)

This is an interative process and requires service loads. A quick check using factored loads with the gross-section moment of inertia of the the 27"x4" concrete beam $I_x = 6561 in^4$ is shown in the load combination results above. By inspection, deflection will not control even if a reduction factor of 100 is used to account for time dependent loading, effective moment of inertia, decreased loading due to service load combinations:

$\frac{\min(L/\delta_{min})}{100} = 225$ **≥** 240

Result Summary:

$$UC_r := \begin{pmatrix} \frac{A_{min}}{A_s} \\ \text{if}(b_w \le s, 0, 2) \\ \frac{V_u}{V_{design}} \\ \frac{M_u}{M_{design}} \end{pmatrix} = \begin{pmatrix} 0.97 \\ 0 \\ 0.52 \\ 0.39 \end{pmatrix}$$ **≤ 1.0**

Composite Beam Embedded Plate Connection Analysis

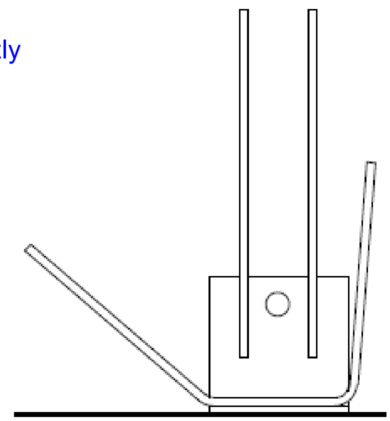
In order for the roof, wall, and floor to act as a composite beam, the embed plates must be capable of transferring the longitudinal shear at the planes of discontinuity. Conservatively use the moment of inertia calculated earlier using only the "chord" areas. The shear along the plane shall be distributed equally to the (5) embedded plates.

$Q_{tc} := A_{tc} \cdot \left(t - 0.5 \cdot t_{r\_e_1} - y\right)$ $\quad Q_{tc} = 1122\, in^3$

$Q_{bc} := A_{bc} \cdot \left(y - 0.5 \cdot t_{f\_act}\right)$ $\quad Q_{bc} = 1122\, in^3$

Should be equal if calculated correctly

$V_{long} := \frac{\max(V_{max}) \cdot Q_{tc}}{I_g} \cdot \frac{L_{act}}{5}$ $\quad \boxed{V_{long} = 4.37\, kip}$ Shear at the embed plates



Weld Connection Analysis

$\boxed{d_w := 0.1875 \cdot in}$ Leg size of fillet weld

$\boxed{F_e := 60 \cdot ksi}$ Weld electrode strength

$\boxed{\Omega_w := 2.0}$ ASD safety factor of welds

$V_{weld} := \sqrt{V_{long}^2 + T_{embed}^2}$ $\quad \boxed{V_{weld} = 7.75 \cdot kip}$ Maximum resultant load on the embed plates

$\boxed{\min\left(\phi V_{n\_rebar}\left(\rho_{f\_act}\right), \phi V_{n\_plate}\right) = 9.46\, kip}$ <--- Embed plate is sufficient

$L_{w\_req} := Ceil\left(\frac{V_{weld} \cdot \Omega_w}{0.6 \cdot F_e \cdot \frac{d_w}{\sqrt{2}}}, 0.25 \cdot in\right)$ $\quad \boxed{L_{w\_req} = 3.25 \cdot in}$ **Required length of 3/16" fillet weld per plate (5" actual)**

No increase for angle of load to the longitudinal axis of weld was assumed.

STEEL SUPPORT BEAM

$L := L_{act}$ Length of the member

$L_b := 0.5 \cdot L$ Unbraced length of the bending member (max concrete pier spacing)

Shape := "W6X12" Enter the name of the shape (Enter fractions of inches following the example: 3-1/2 for 3.5")

$w_{DL} := \frac{W_{DL}(B_{act}, L_{act}, \rho_{lw}, \rho_{lw}, \rho_{f\_act})}{2 \cdot L_{act}}$ $w_{DL} = 1403 \cdot plf$

$w_{LL} := max[LL_f, max(LL_r, p_f), 0.75 \cdot (LL_f + max(p_f, LL_r))] \cdot (0.5 \cdot B_{act})$ $w_{LL} = 1313 \cdot plf$

$w_{WL} := P_{Z3\_neg}(B_{act}) \cdot (0.5 \cdot B_{act})$ $w_{WL} = -546 \cdot plf$

▶ Material and Section Properties Lookup

Member Analysis - LC 1 [DL+max(LL,SL)]

***** Alternative to the deep beam wall analysis, a steel beam may span piers and provide support for the bearing walls. *****

Support Locations

$x_{r1} := 0$ Support location 1

$x_{r2} := 0.5 \cdot L$ Support location 2

$x_{r3} := L$ Support location 3

Distributed Loads | Start | End

$w_1 := w_{DL} + w_{LL} + w$ $x_{w1\_s} := 0$ $x_{w1\_e} := L$

▶ Shear, Bending, and Deflection Analysis
▶ Allowable Tension, Compression, Shear and Moment Calculation

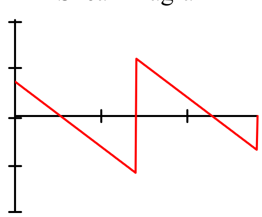
$V_{max_1} = 11.932 \cdot kip$

$M_{max_1} = 200 \cdot kip \cdot in$

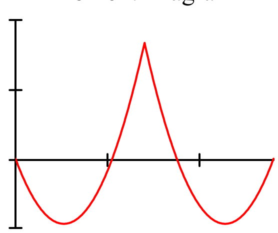
$\delta_{max_1} = 0.096 \cdot in$

$L/\delta_{min_1} = 878 \geq 240$

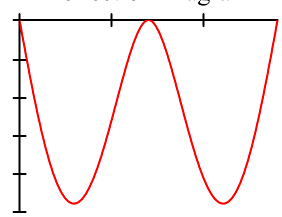
Shear Diagram



Moment Diagram



Deflection Diagram



$R_{r1_1} = 7.16 \cdot kip$ $R_{r2_1} = 23.86 \cdot kip$ $R_{r3_1} = 7.16 \cdot kip$

$M_{a\_x} = 204 \cdot kip \cdot in \geq M_{max_1} = 200 \cdot kip \cdot in$

$V_a = 24.9 \cdot kip \geq (V_{max_1}) = 11.9 \cdot kip$

Member Analysis - LC 2 [0.6DL+0.6WL]

Support Locations

$x_{r1} := 0$ Support location 1

$x_{r2} := 0.5 \cdot L$ Support location 2

$x_{r3} := L$

$x_{r4} := 0$

Distributed Loads

	Start	End
$w_1 := 0.6 \cdot (w_{DL} + w_{WL} + w)$	$x_{w1\_s} := 0$	$x_{w1\_e} := L$

▶ Shear, Bending, and Deflection Analysis

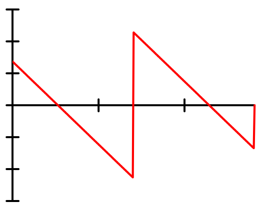
$V_{max_2} = 2.281 \cdot kip$

$M_{max_2} = 38 \cdot kip \cdot in$

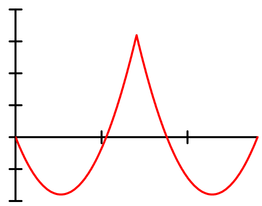
$\delta_{max_2} = 0.018 \cdot in$

$L/\delta_{min_2} = 4595$ ≥ 240

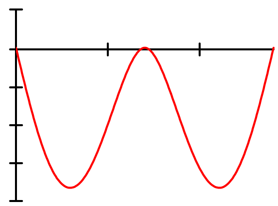
Shear Diagram



Moment Diagram



Deflection Diagram



$R_{r1_2} = 1.37 \cdot kip$ $R_{r2_2} = 4.56 \cdot kip$

$M_{a\_x} = 204 \cdot kip \cdot in$ ≥ $M_{max_2} = 38 \cdot kip \cdot in$

$V_a = 24.9 \cdot kip$ ≥ $V_{max_2} = 2.3 \cdot kip$