Chapter 2

Highway Bridge Superstructure Systems:

LRFD Approaches to Design and Analysis

Problems

Problem 2.1

Fig. 2.1-1a shows the cross section of a simply supported two-lane highway bridge spanning 60 ft. The concrete deck is 8.5 in. thick including ½-in. thick integral wearing surface. It is supported over five W30 x 130 rolled steel girders spaced at 7 ft 6 in. on center. A typical haunch is 2-in. thick over the steel girder flanges. The traffic barriers would be placed after the deck has hardened sufficiently. The cross sectional details of the concrete traffic barriers are shown in Fig. 2.1-1b. The bridge design requires a provision of 25-psf dead load for future wearing surface (FWS). Assume f'_c = 4 ksi for concrete deck and F_y = 50 ksi for the steel girders. The bridge has no skew. Assume unshored construction.

Assuming <u>noncomposite</u> construction, determine if the girders satisfy service limit states for the bridge.

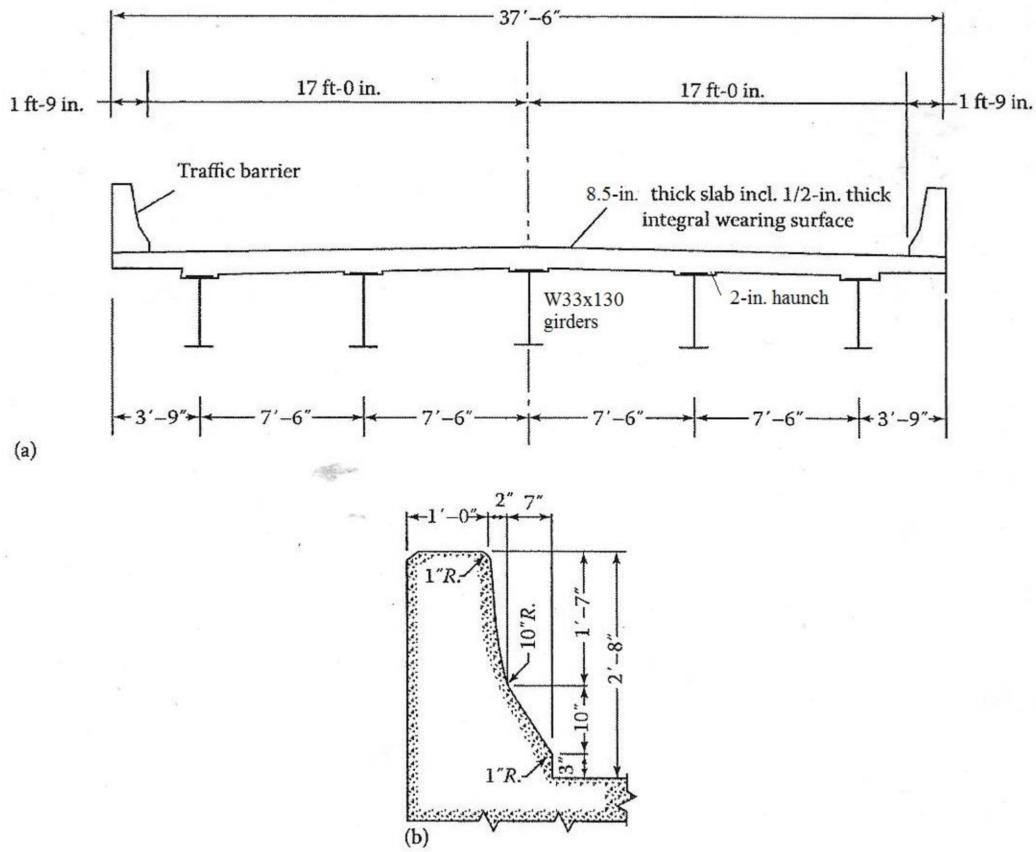

Fig. 2.1-1a and 1b.

Problem 2.2

For the two-lane bridge escribed in Prob. 2.1, check if it the W30 x 130 steel girders would satisfy the service load criteria if the construction would be composite and unshored. All other conditions remain the same.

Problem 2.3

For the two-lane, noncomposite bridge described in Prob. 2.1, determine the approximate lightest weight rolled steel wide flange section to satisfy Owner's Optional Deflection criteria.

Solutions to Problems
Chapter 2

Problem 2.1

Fig. 2.1-1a shows the cross section of a simply supported two-lane highway bridge spanning 60 ft. The concrete deck is 8.5 in. thick including ½-in. thick integral wearing surface. It is supported over five W30 x 130 rolled steel girders spaced at 7 ft 6 in. on center. A typical haunch is 2-in. thick over the steel girder flanges. The traffic barriers would be placed after the deck has hardened sufficiently. The cross sectional details of the concrete traffic barriers are shown in Fig. 2.1-1b. The bridge design requires a provision of 25-psf dead load for future wearing surface (FWS). Assume f'_c = 4 ksi for concrete deck and F_y = 50 ksi for the steel girders. The bridge has no skew. Assume unshored construction.

Assuming <u>noncomposite</u> construction, determine if the girders satisfy service limit states for the bridge.

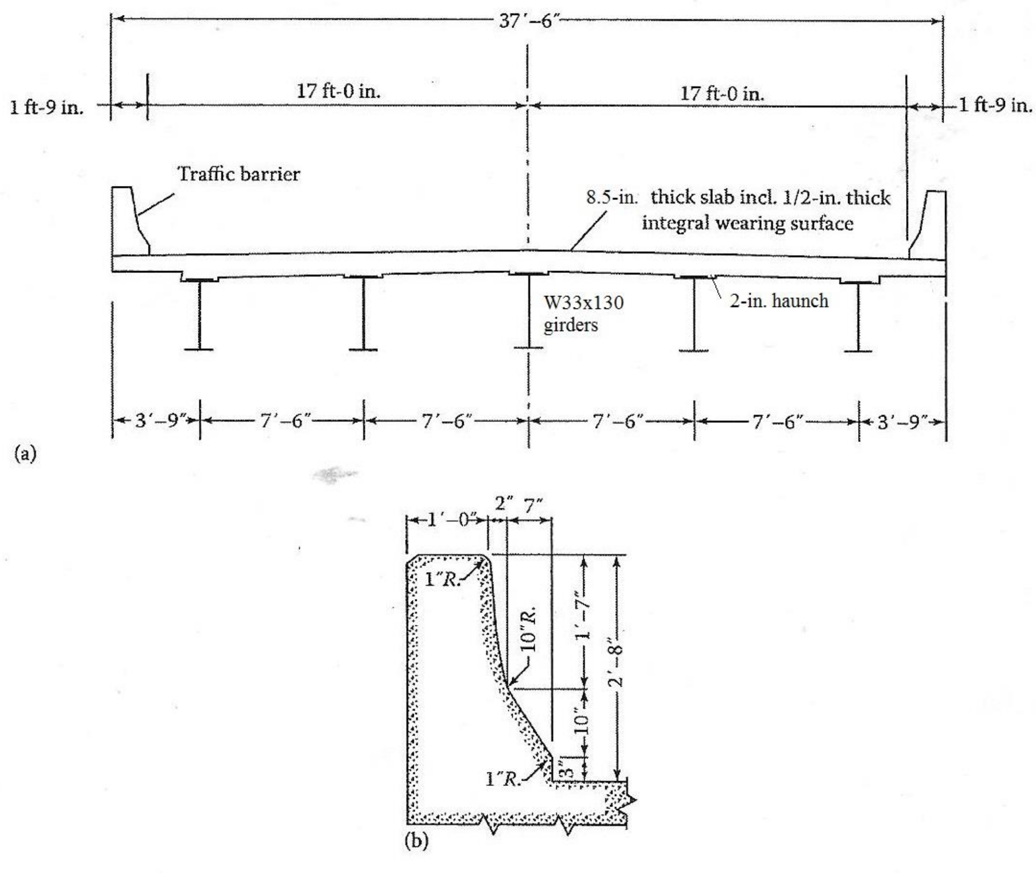

Fig. 2.1-1a and 1b.

Solution

Check for Service Limit State for steel girder bridges (Art 6.5.2, A6.10.4)

Art. 6.10.4.1: Compliance with Art. 2.5.2.6 is required.

1. Optional live load-deflection deflection control:

Since no deflection criteria are specified, optional deflection criteria (Art. 2.6.2.6.2) will be used in this problem. Span L = 60 ft.

$$\text{Allowable deflection due to vehicular load, } \Delta_{LL} = \frac{span}{800} = \frac{60(12)}{800} = 0.9 \; in.$$

Art. 3.6.1.3.2: Calculated deflection will be taken as the larger of the following as discussed in Sec. 2.6.3 (Ch. 2):

1. Deflection due to design truck alone. Dynamic load allowance will be applied to this deflection.
2. Deflection due to 25 percent of the deflection due to the design truck *plus* deflection due to lane load. Dynamic load allowance is not applied to the deflection due to the lane load.

Calculate deflection due to vehicular live load.

Live load-deflections would be computed for the governing loading conditions – separately for the design truck and lane (Fig. P2.1-2) – by the usual methods of computing deflections. Formulas for relevant load cases given in AISC Manual of Steel Construction [AISC 2006, 13th ed.] would be used (see discussion in Ch.2, Sec. 2.6.6). Referenced formulas used in calculations are taken from Ch. 2. For the noncomposite beams used in this problem, the moment of inertia, *I*, of the steel section would be used for calculating deflections.

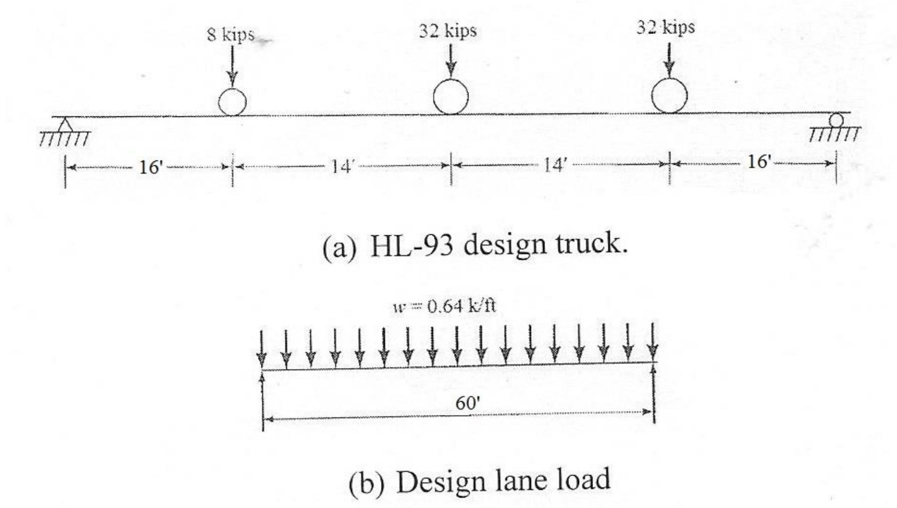

Fig. 2.1.-2 HL-93 live loads: (a) design truck, (b) lane load.

Deflection due to design truck

As discussed in Sec. 2.6.6, in the case of the design truck loading it is difficult, although theoretically possible, to determine the exact positions of the three axles that will result in maximum deflection. As a practical matter, however, the maximum deflection in the girder would be computed by placing the middle 32-kip load at the midspan and the other two loads (8-kip and 32-kip loads) at 14 ft from the midspan, one on the left and the other on the right side of the midspan (Fig. P2.1-3). This load case can be split in two separate cases as shown in AISC's *Steel Construction Manual*:

a. Load Case 7 for the centrally placed point load (Fig. P2.1-3a)
b. Load Case 8 for point loads placed off the midspan (Fig. P2.1-3b and c)

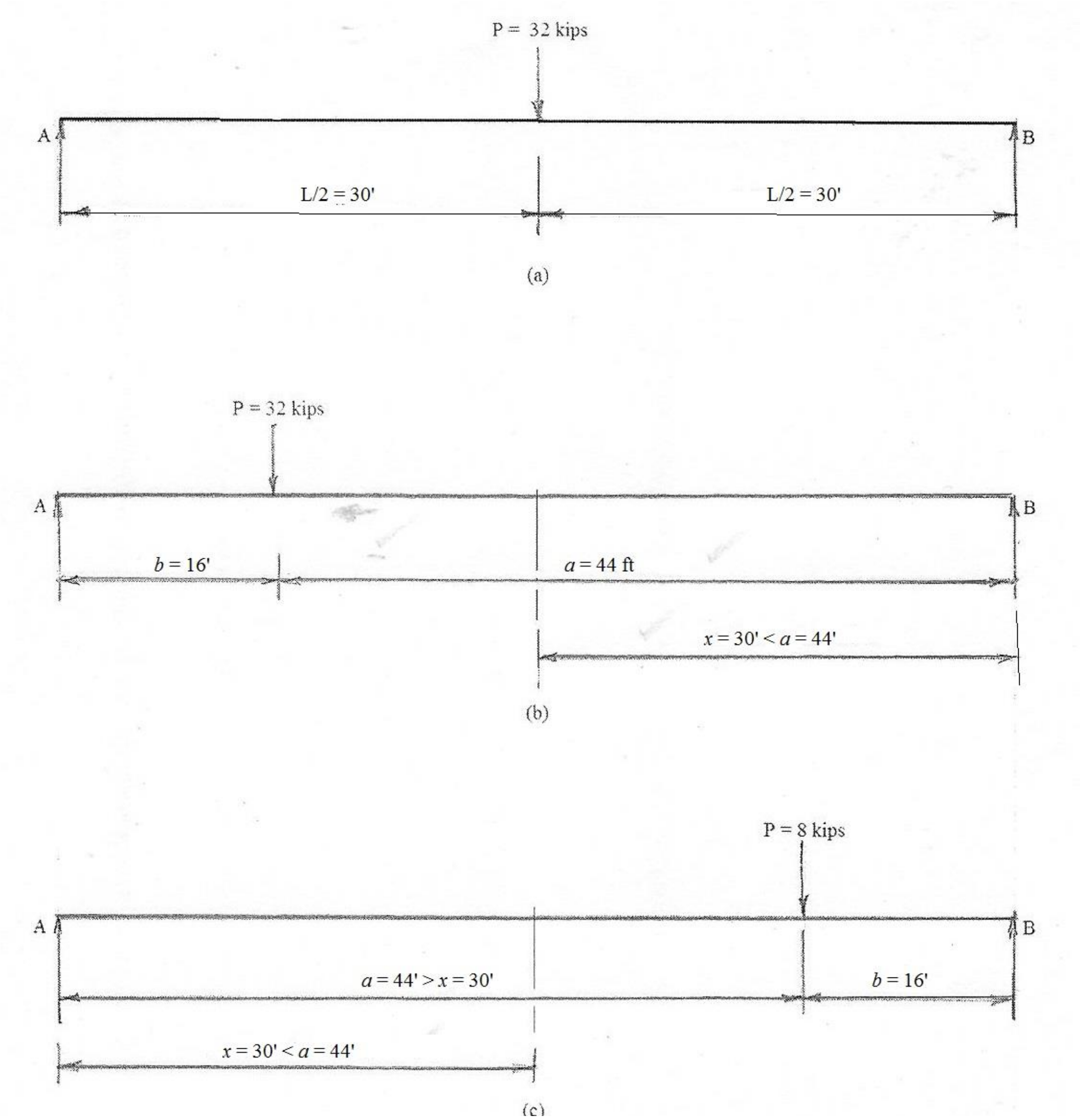

Fig. P2.1-3 (a) point load at midspan, (b) and (c) point loads off midspan.

For the centrally placed point load P (Load Case 7), the maximum deflection occurs at the center, which can be calculated from Eq. 2.1-1:

$$\Delta_c = \frac{PL^3}{48EI} \tag{2.1-1}$$

where

Δ_c = deflection at midspan due to point load
L = span = 60 ft
E = modulus of elasticity = 29,000 ksi
I = moment of inertia = 6710 in.^{4} (W33 x 130), noncomposite construction

The axle loads should be multiplied by (1 + *IM*) = (1 + 0.33) = 1.33 to account for dynamic allowance. The resulting axle loads are:

P_1 = 32(1.33) = 42.56 kip at midspan

P_2 = 32(1.33) = 42.56 kip at 16 ft from the left support (x measured from B)

P_3 = 8(1.33) = 10.64 kip at 16 ft from the right support (x measured from A)

Substitute P = 42.56 kip, L = 60 ft, E = 29,000 ksi, and I = 5900 in.^{4} in Eq. 2.1. The deflection at the midspan is

$$\Delta_c = \frac{42.56(60)^3(12)^3}{48(29{,}000)(6710)} = 1.70\ in.$$

For the remaining axle loads which are placed off-center on the span, Load Case 8 would be used.

For a point load P placed at a distance a from the right support, the deflection at a distance x (when $x < a$) from the right support (x measured from the right support B Fig. P2.1-3b) can be calculated from Eq. 2.1-2:

$$\Delta_x = \frac{Pbx(L_2 - b^2 - x^2)}{6EIL} \tag{2.1-2}$$

Eq. 2.1-2 is used to calculate deflections at the midspan due to the two off-center point loads (the 10.56-kip and 42.56-kip loads, including *IM*). Substitute $P = P_2 + P_3$ = (42.56 + 10.64 = 53.2) kip, $x = L/2$ = 30 ft, b = 16 ft, E = 29,000 ksi, I = 6710 in^4 and L = 60 ft in Eq. 2.1-2. The deflection at the midspan (x = 30 ft) is

$$\Delta_{x=30\ ft} = \frac{(53.2)(16)(30)[(60)^2 - (16)^2 - (30)^2]}{6(29{,}000)(6710)(60)}(12)^3 = 1.54\ in.$$

Total deflection due to the design truck, $\Delta_c + (\Delta_{x\,=\,30\text{ ft}})$ = 1.70 + 1.54 = 3.24 in.

Deflection due to lane load

For the case of lane loading (uniform load), the maximum deflection is calculated from Eq. 2.1-3:

$$\Delta_{lane} = \frac{5wL^4}{384EI} \tag{2.1-3}$$

where

w = 0.64 k/ft

L = span = 60 ft

Substituting these values along with E = 29,000 ksi and I = 6710 in.^{4} in Eq. 2.1-3, the maximum deflection due to the lane load is

$$\Delta_{lane.max} = \frac{5(0.64)(60)^4(12)^3}{384(29000)(6710)} = 0.96\ in.$$

Apply live load distribution factors (LLDF) for deflection to the deflection values calculated above.

$$\text{LLDF for deflection} = N_L/N_b$$

where N_L = number of design lanes = 2

N_b = number of girders supporting the deck = 5.

$$\text{Therefore, LLFD for live load deflection} = N_L/N_b = 2/5 = 0.4$$

$$\text{Maximum deflection in the girder due to design truck, } \Delta_{\text{truck}} = (0.4)\ (3.24) = 1.30 \text{ in.}$$

$$\text{Maximum deflection due to the lane load, } \Delta_{lane} = 0.4(0.96) = 0.38 \text{ in.}$$

Compare deflections.

$$\Delta_{\text{truck}} = 1.30 \text{ in}$$

$$0.25\Delta_{\text{truck}} + \Delta_{\text{lane}} = 0.25(1.30) + 0.38 = 0.71 \text{ in.}$$

$$\Delta_{\text{truck}} = 1.30 \text{ in.} > (0.25\Delta_{truck} + \Delta_{lane}) = 0.71 \text{ in.}$$

$$\text{Therefore, } \Delta_{truck} = 1.30 \text{ in. controls.}$$

$$\text{Allowable live load deflection, } \Delta_{allowable} = \frac{L}{800} = \frac{60\ x\ 12}{800} = 0.90\ in.$$

$$\Delta_{truck} = 1.30 \text{ in.} > \Delta_{allowable} = 0.9 \text{ in.}$$

The optional deflection criteria are <u>not</u> satisfied.

Optional criteria for Span-to-Depth Ratio (Art. 2.5.2.6.3)

The Owner may invoke the optional criteria for span-to-depth ratio as specified in Art. 2.5.2.6.3/ Table 2.5.2.6.3-1 (see discussion in Ch. 2, Table 2.5), which requires this ratio to be equal to at least 0.033L. For this problem,

$$0.033(60)\ (12) = 23.76 \text{ in.}$$

$$\text{Depth of girder, } d = 33.1 \text{ in.} > 23.76 \text{ in., OK.}$$

Span-to-Depth Ratio is satisfied.

2. Permanent Deformations (Art. 6.10.4.2)

The purpose of check is to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. For noncomposite sections, compliance with Eq. 2.1-4 (A6.10.4.2.2-3) is required:

$$f_f + \frac{f_\ell}{2} \leq 0.8 R_h F_{yf} \qquad \text{(2.1-4/A6.10.4.2.2-3)}$$

f_f = flange stress at the section under consideration due to Service II Loads calculated without consideration of flange lateral bending
f_ℓ = flange lateral buckling stress at the section under consideration due to Service II Loads determined as specified in Art. 6.10.1.6 (ksi)
≈ 0 for wide flange section

For Service II Loads, the maximum unfactored moments are as follows is (see solution to Prob. 3.1 for details):

$$M_{\mathrm{DC}} = M_{DC1} + M_{DC2} = 474.3 + 90.9 = 565.2 \text{ kip-ft}$$
$$M_{\mathrm{DW}} = 84\ 6 \text{ kip-ft}$$
$$M\,(LL + IM) = 908 \text{ kip-ft}$$

For Service II Loads, the maximum factored moments are obtained as follows*:

$$\begin{aligned} M_u &= \eta[1.0M_{DC} + 1.0M_{DW} + 1.3(LL + IM)] \\ &= (1.0)[1.0(565.2)\ 1.0(84.6) + 1.3(908)] \\ &= 1830.2 \text{ kip-ft} \end{aligned}$$

Section modulus of W33 x 130, S_x = 406 in.^{3}

$$f_f = \frac{M_u}{S_x} = \frac{1830.2(12)}{406} = 54.1\ ksi$$

$$0.8R_hF_{yf} = 0.8(1.0)(50) = 40 \text{ ksi}$$

$$f_f = 541 \text{ ksi} > 0.8R_hF_{yf} = 40 \text{ ksi, } \textbf{NG}.$$

The condition for permanent deformations is not satisfied.

*Note: These moment values would have to be determined before the next step.

Problem 2.2

For the two-lane bridge escribed in Prob. 2.1, check if it the W30 x 130 steel girders would satisfy the service load criteria if the construction would be composite and unshored. All other conditions remain the same.

Solution:

Check for Service Limit State for steel girder bridges (Art 6.5.2, A6.10.4)

Art. 6.10.4.1: Compliance with Art. 2.5.2.6 is required.

3. Optional live load-deflection deflection control:

Since no deflection criteria are specified, optional deflection criteria (Art. 2.6.2.6.2) will be used in this problem. Span L = 60 ft.

$$\text{Allowable deflection due to vehicular load, } \Delta_{LL} = \frac{span}{800} = \frac{60(12)}{800} = 0.9\ in.$$

Art. 3.6.1.3.2: Calculated deflection will be taken as the larger of the following as discussed in Sec. 2.6.3 (Ch. 2):

3. Deflection due to design truck alone. Dynamic load allowance will be applied to this deflection.
4. Deflection due to 25 percent of the deflection due to the design truck *plus* deflection due to lane load. Dynamic load allowance is not applied to the deflection due to the lane load.

Calculate deflection due to vehicular live load.

Live load-deflections would be computed for the governing loading conditions – separately for the design truck and lane (Fig. P2.2-1) – by the usual methods of computing deflections. Formulas for relevant load cases given in AISC Manual of Steel Construction would be used (see discussion in Ch.2, Sec. 2.6.6). Referenced formulas used in calculations are taken from Ch. 2. For the composite beams used in this problem, the moment of inertia, I_c, of the composite steel section would be used for calculating deflections.

Determination of the moment of inertia of composite section with W33 x 130 steel girders, and the short-term and long-term section properties.

Calculate the moment of inertia, I_c, of the composite section (Fig. P2.2-2). Calculations are shown in the following table. All perpendicular distances are measured from the bottom of steel girder.

Calculate the transformed area of slab, A_{tr}. The effective width of slab,

b_e = tributary width of slab = 7 ft 6 in. = 90 in.

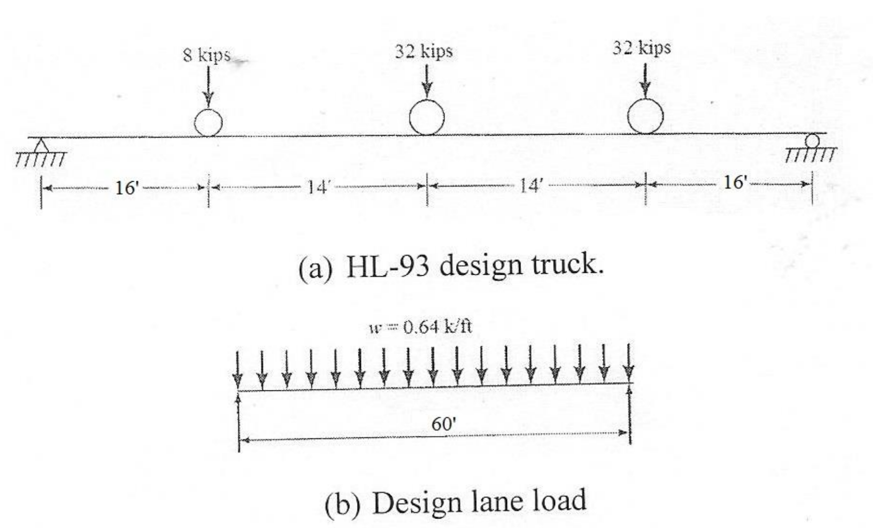

Fig. P2.2-1 HL-93 live loads: (a) design truck, (b) lane load.

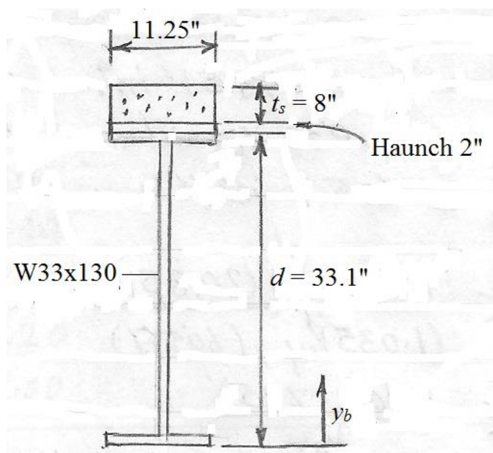

Fig. P2.2-2 Cross section of composite steel girder.

For f_c' = 4000 psi, the modular ratio, n = 8

For short-term section properties, use n = 8

For long-term section properties, use 3n = 3(8) = 24

For the short-term properties (to be used for calculating deflections due to the live load), the transformed area of the deck slab, A_{tr}, is calculated to be,

$$A_{tr} = \frac{b_e t_s}{n} = \frac{(90)(8)}{8} = 90\ in.^2$$

For the long-term properties (to be used for checking for permanent deformations), the transformed area of the deck slab, A_{tr}, is calculated to be,

$$A_{tr} = \frac{b_e t_s}{3n} = \frac{(90)(8)}{3(8)} = 90\ in.^2$$

Calculations for the short-term properties of the composite section are shown in the following table:

Short-term Section Properties

Component	A, in.^{2}	y_b, in.	Ay_b, in.^{3}	$(y_b - \bar{y}_b)$, in.	$A(y_b - \bar{y}_b)^2$, in.^{4}	I_o, in.^{4}
Conc. slab	90.0	39.1	3519	6.73	4076	480
Steel girder	38.3	16.55	634	15.82	9585	6710
Σ	128.3		4153		13,661	7190

$$\bar{y}_b = \frac{4153}{(128.3)} = 32.37\ in.$$

$$I_{comp} = I_c = I_o + A\bar{d}^2 = 7190 + 13{,}661 = 20{,}851\text{ in.}^4$$

Perpendicular distance from the NA of composite section to top of steel,

y_t = depth of steel girder - $\bar{y}_b$ = 33.1 – 32.37 = 0.73 in.

Section modulus at top of steel,

$$S_{ST}^t = \frac{I_c}{y_t} = \frac{20{,}851}{0.73} = 28{,}563\ in.^3$$

Section modulus at bottom of steel,

$$S_{ST}^b = \frac{I_c}{y_b} = \frac{20{,}851}{32.37} = 644\ in.^3$$

Long-term Section Properties

Component	A, in.^{2}	y_b, in.	Ay_b, in.^{2}	$(y_b - \bar{y}_b)$, in.	$A(y_b - \bar{y}_b)^2$, in.^{4}	I_o, in.^{4}
Conc. slab	30.0	39.1	1173	12.64	4793	160
Steel girder	38.3	16.55	634	-9.91	3761	6710
Σ	68.3		1807		8554	6870

$$\bar{y}_b = \frac{1807}{(68.3)} = 26.46\ in.$$

$$I_{comp} = I_c = I_o + A(y_b - \bar{y}_b)^2 = 6870 + 8554 = 15{,}424 \text{ in.}^4$$

Perpendicular distance from the NA of composite section to top of steel,

$$y_t = \text{depth of steel girder} - \bar{y}_b = 33.1 - 24.46 = 6.64 \text{ in.}$$

Section modulus at top of steel,

$$S_{ST}^t = \frac{I_c}{y_t} = \frac{15{,}424}{6.64} = 2323 \; in.^3$$

Section modulus at bottom of steel,

$$S_{ST}^b = \frac{I_c}{y_b} = \frac{15{,}424}{26.46} = 583 \; in.^3$$

Deflection due to design truck

As discussed in Sec. 2.6.6, in the case of the design truck loading it is difficult, although theoretically possible, to determine the exact positions of the three axles that will result in maximum deflection. As a practical matter, however, the maximum deflection in the girder would be computed by placing the middle 32-kip load at the midspan and the other two loads (8-kip and 32-lip loads) 14 ft from the midspan, one on the left and the other on the right side of the midspan (Fig. 2.2-3). This load case can be split in two separate cases as shown in AISC Steel Construction Manual:

a. Load Case 7 for the centrally placed point load (Fig. P2.2-3a)
b. Load Case 8 for point loads placed off the midspan (Fig. P2.2-3b and c)

For the centrally placed point load P (Load Case 7), the maximum deflection occurs at the center which can be calculated form Eq. 2.2-1:

$$\Delta_c = \frac{PL^3}{48EI_c} \qquad (2.2\text{-}1)$$

where

Δ_c = deflection at midspan due to point load
L = span = 60 ft
E = modulus of elasticity = 29,000 ksi
I_c = moment of inertia of composite section = 20,851 in.^{4} (composite construction)

The axle loads should be multiplied by (1 + IM) = (1 + 0.33) = 1.33 to account for dynamic allowance. The resulting axle loads are:

P_1 = 32(1.33) = 42.56 kip at midspan

P_2 = 32(1.33) = 42.56 kip at 16 ft from the left support (x measured from B)

P_3 = 8(1.33) = 10.64 kip at 16 ft from the right support (x measured from A)

Substitute P = 42.56 kip, L = 60 ft, E = 29,000 ksi, and I_c = 20,851 in.^{4} (short-term composite section property).in Eq. 2.2-1. The deflection at the midspan is

$$\Delta_c = \frac{42.56(60)^3(12)^3}{48(29{,}000)(20{,}851)} = 0.55\ in.$$

For the remaining axle loads which are placed off-center on the span, Load Case 8 would be used.

For a point load P placed at a distance a from the right support, the deflection at a distance x (when $x < a$) from the right support (x measured from the right support B Fig. P2.2-3b) can be calculated from Eq. 2.2-2:

$$\Delta_x = \frac{Pbx(L_2 - b^2 - x^2)}{6EIL} \tag{2.2-2}$$

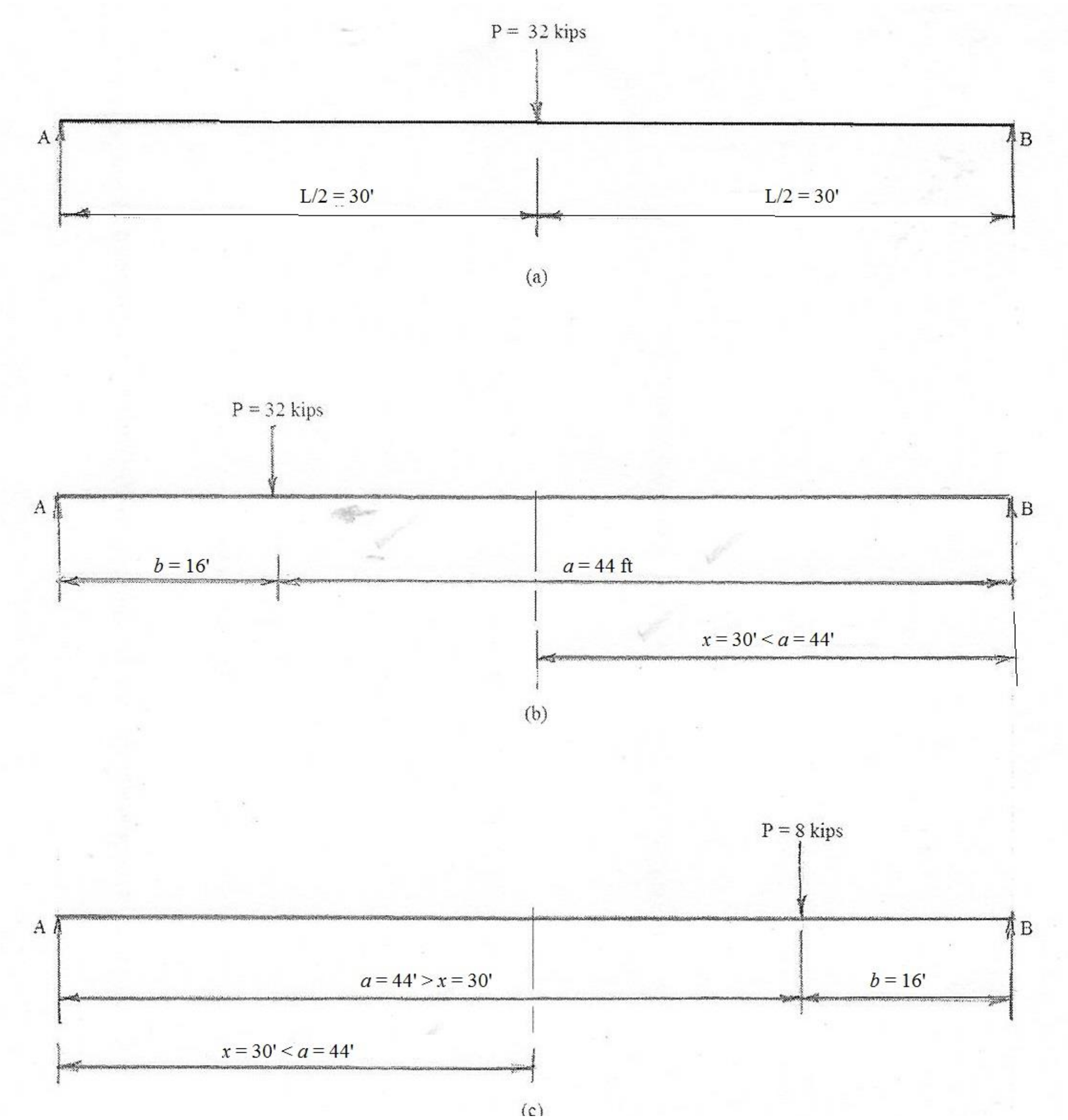

Fig. P2.2-3 (a) point load at midspan, (b) and (c) point loads off midspan.

Eq. 2.2-2 is used to calculate deflections at the midspan due to the two off-center point loads (the 10.64-kip and 42.56-kip loads, including *IM*). Substitute $P = P_2 + P_3 = 42.56 + 10.56 = 53.2$ kip, $x = L/2 = 30$ ft, $b = 16$ ft, $E = 29{,}000$ ksi, $I_c = 20{,}851$ in.^{4} and $L = 60$ ft in Eq. 2.2-2. The deflection at the midspan ($x = 30$ ft) is

$$\Delta_{x=30\,ft} = \frac{(53.2)(16)(30)[(60)^2 - (16)^2 - (30)^2]}{6(29{,}000)(20{,}851)(60)}(12)^3 = 0.50\ in.$$

Total deflection due to the design truck, $\Delta_c + (\Delta_{x\,=\,30\,ft}) = 0.55 + 0.50 = 1.05$ in.

Deflection due to lane load

For the case of lane loading (uniform load), the maximum deflection is calculated from Eq. 2.2-3 (Case 1, AISC 2011):

$$\Delta_{lane} = \frac{5wL^4}{384EI_c} \tag{2.2-3}$$

where

$w = 0.64$ k/ft
L = span = 60 ft

Substituting these values along with $E = 29{,}000$ ksi and $I_c = 20{,}851$ in.^{4} in Eq. 2.2-3, the maximum deflection due to lane load is

$$\Delta_{lane.max} = \frac{5(0.64)(60)^4(12)^3}{384(29000)(20{,}851)} = 0.31\ in.$$

Apply live load distribution factors (LLDF) for deflection to the deflection values calculated above.

LLDF for deflection = N_L/N_b

where N_L = number of design lanes = 2

N_b = number of girders supporting the deck = 5.

Therefore, LLDF for deflection = $N_L/N_b = 2/5 = 0.4$

Apply LLDF for deflection = 0.4.

Maximum deflection in the girder due to design truck, $\Delta_{truck} = (0.4)(1.05) = 0.42$ in.

Maximum deflection in the girder due to the lane load, $\Delta_{lane} = 0.4(0.31) = 0.12$ in.

Compare deflections.

$\Delta_{truck} = 0.42$ in

$0.25\Delta_{truck} + \Delta_{lane} = 0.25(0.42) + 0.12 = 0.23$ in.

$$\Delta_{\text{truck}} = 0.42 \text{ in.} > (0.25\Delta_{truck} + \Delta_{lane}) = 0.23 \text{ in.}$$

$$\text{Therefore, } \Delta_{truck} = 0.31 \text{ in. controls.}$$

$$\text{Allowable live load deflection, } \Delta_{allowable} = \frac{L}{800} = \frac{60\ x\ 12}{800} = 0.90\ in.$$

$$\Delta_{truck} = 0.23 \text{ in.} < \Delta_{allowable} = 0.9 \text{ in.}$$

The optional deflection criteria are satisfied.

Optional criteria for Span-to-Depth Ratio (Art. 2.5.2.6.3)

The Owner may invoke the optional criteria for span-to-depth ratio as specified in Art. 2.5.2.6.3/ Table 2.5.2.6.3-1 (see discussion in Ch. 2, Table 2.5), which requires this ratio to be equal to at least $0.04L$. For this problem,

$$0.04(60)(12) = 28.8 \text{ in.}$$

Overall depth of the composite section,

$$D_t = \text{slab thickness + haunch thickness + depth } d \text{ of W33 x 130}$$

$$= 8 + 2 = 33.1 = 43.1 \text{ in.} > 28.8 \text{ in.}$$

Span-to-depth Ratio is satisfied.

4. Permanent Deformations (Art. 6.10.4.2)

The purpose of check is to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. For composite sections, compliance with Eq. 2.2-4 (A6.10.4.2.2-3) is required:

$$f_f + \frac{f_\ell}{2} \leq 0.95R_hF_{yf} \qquad \text{(2.2-4/A6.10.4.2.2-2)}$$

f_f = flange stress at the section under consideration due to Service II Loads calculated without consideration of flange lateral bending
f_ℓ = flange lateral buckling stress at the section under consideration due to Service II Loads determined as specified in Art. 6.10.1.6 (ksi)
= 0 for composite sections
F_{yf} = yield strength of the flanges of the steel girder = 50 ksi

$$0.95R_hF_{yf} = 0.95(1.0)(50) = 47.5 \text{ ksi}$$

Therefore, it is required that the flange stress, f_f, be < 47.5 ksi.

Calculate the flange stresses as follows (see discussion on p. 900-901 of the textbook):

For M_{DC1}, use noncomposite section properties (S_x = 406 in.^{3} for W33 x 130)
For M_{DC2} and M_{DW}, use long-term composite section properties.
For 1.3M_{LL+IM}, use short-term composite section properties.

From solution of Prob. 3.1, the following values of moments are obtained:

$$M_{DC1} = 474.3 \text{ kip-ft}$$
$$M_{DC2} + M_{DW} = 90.9 + 84.6 = 175.5 \text{ kip-ft}$$
$$1.3M_{LL+IM} = 1.3(908) = 1180.4 \text{ kip-ft}$$

Compressive stress in the top flange of composite section:

$$f_c = \frac{474.3(12)}{406} + \frac{175.5(12)}{2323} + \frac{1180.4(12)}{28,563} = 14.02 + 0.91 + 0.50 = 15.43\ ksi$$

$$f_c = \frac{474.3(12)}{406} + \frac{175.5(12)}{583} + \frac{1180.4(12)}{644} = 14.02 + 3.61 + 22.00 = 39.63\ ksi$$

$$f_f = 39.63 \text{ ksi} > 0.95R_hF_{yf} = 47.5 \text{ ksi, } \textbf{OK}.$$

The conditions for permanent deformations are satisfied.

Problem 2.3

For the two-lane, noncomposite bridge described in Prob. 2.1, determine the approximate lightest weight rolled steel wide flange section to satisfy Owner's Optional Deflection criteria.

Solution:

From the solution to Prob. 2.1, governing value of live load deflection,

$$\Delta_{truck} = 1.30 \text{ in. (controls)}$$

$$\text{Allowable live load deflection, } \Delta_{allowable} = \frac{L}{800} = \frac{60 \ x \ 12}{800} = 0.90 \ in.$$

$$\Delta_{truck} = 1.30 \text{ in.} > \Delta_{allowable} = 0.9 \text{ in., } \mathbf{NG}.$$

The optional deflection criteria are <u>not</u> satisfied.

Since the bridge girders are noncomposite, the deflection is inversely proportional to the moment of inertia I_x of the steel girder, W30 x 130, I_x = 6710 in.^{4} The required moment of inertia of the steel girder to have $\Delta_{truck} = \Delta_{allowable}$ = 0.9 in can be calculated by proportion. Thus,

$$I_{x,reqd} = \frac{1.3(6710)}{0.9} = 9692 \ in.^4$$

From AISC Steel Construction Manual, the lightest wide flange section for which $I_x \geq$ 9692 in.^{4} is W40 x 149 for which I_x = 9800 in.^{4} > $I_{x,\,reqd}$ = 9692 in.^{4}

W40 x 149 girders (noncomposite construction) <u>would satisfy</u> optional deflection criteria.

Important Note: This problem addresses only the optional deflection requirements for the bridge (a Service Limit State). It should not be erroneously inferred that W40 x1 49 girders would also satisfy requirements for other limit states for the bridge. For example, W40 x 149 noncomposite girders may not satisfy Strength I Limit State.