7.10.4 Prestressed Girder Bridge

Problem Statement Design the simply supported pretensioned prestressed concrete girder bridge of Figure E7.4-1 with a span length of 30 480 mm center to center of bearings for a *HL*-93 live load. The roadway width is 13 420 mm curb-to-curb. Allow for a future wearing surface of 75-mm thick bituminous overlay and use the concrete deck design of Example Problem 7.10.1 (f'_c = 30 MPa). Follow the beam and girder bridge outline in Section 5-Appendix A5.3 of the AASHTO (1994) LRFD Bridge Specifications. Use f'_{ci} = 40 MPa, f'_c = 55 MPa, f_y = 400 MPa, and 1860 MPa, low-relaxation 12.70 mm, seven-wire strands.

A. ***Develop General Section*** The bridge is to carry interstate traffic in Virginia over a single-track railroad with minimum vertical clearance of 7000 mm (Fig. E7.4-1).

B. ***Develop Typical Section*** Use a precast pretensioned Nebraska University girder (Green and Tadros, 1994) made composite with the deck. These are relatively new girder shapes and have been developed in metric to incorporate the best features of strength, efficiency, and constructability (Fig. E7.4-2). They may not be available in all regions of the United States. If other shapes are used, the procedures given in this example are still valid. The only difference is that different values are given for the section properties.

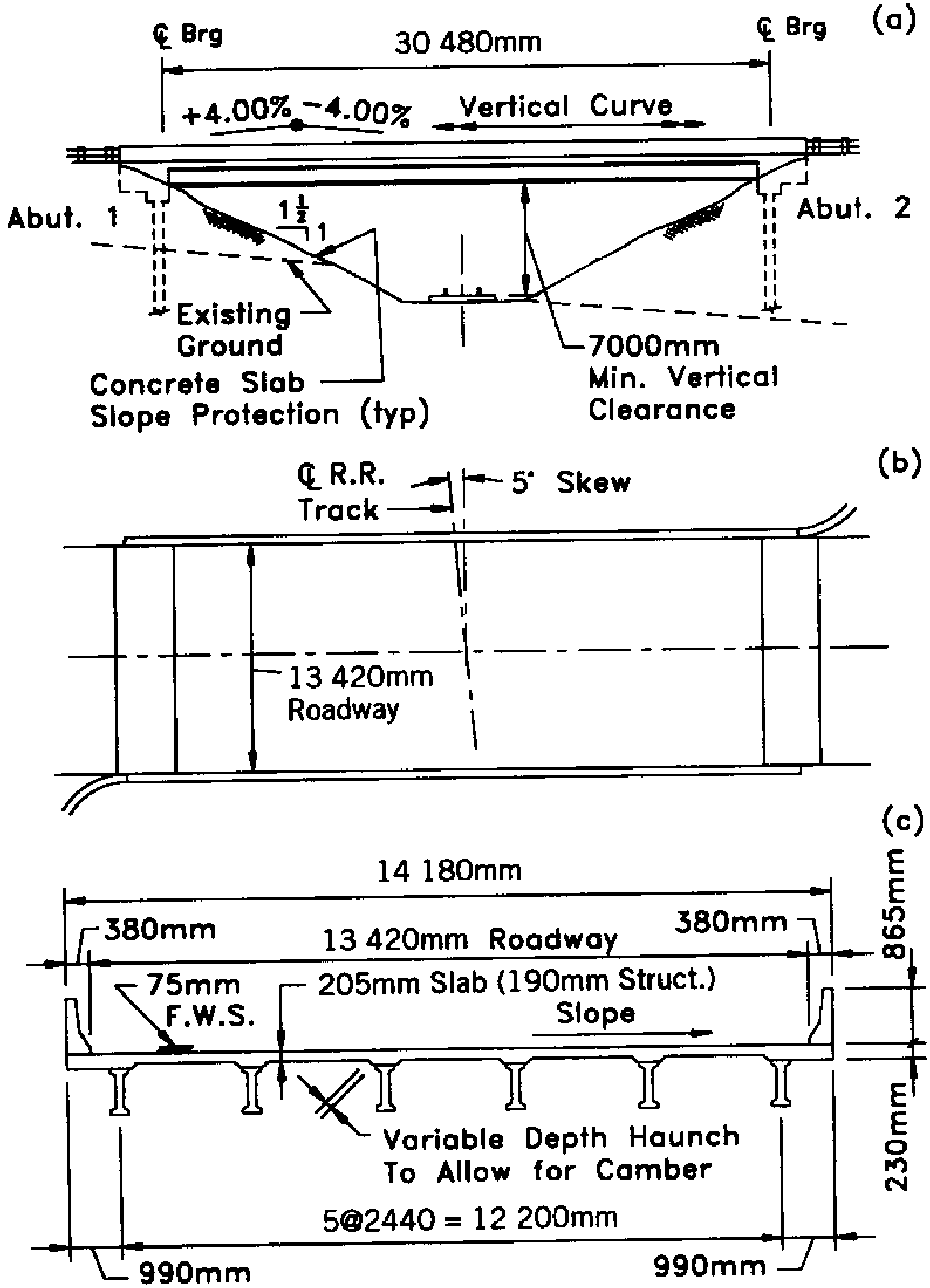


Fig. E7.4-1 Prestressed concrete girder bridge design example (a) elevation, (b) plan, and (c) section.

Example 7.10.4 — Prestressed Girder Bridge
Barker & Puckett

Minimum Thicknesses [A 5.14.1.2.2]
of Precast Girder

Top Flange ≥ 50 mm, OK

Web, non-post-tensioned ≥ 125 mm, OK

Bottom Flange ≥ 125 mm, OK

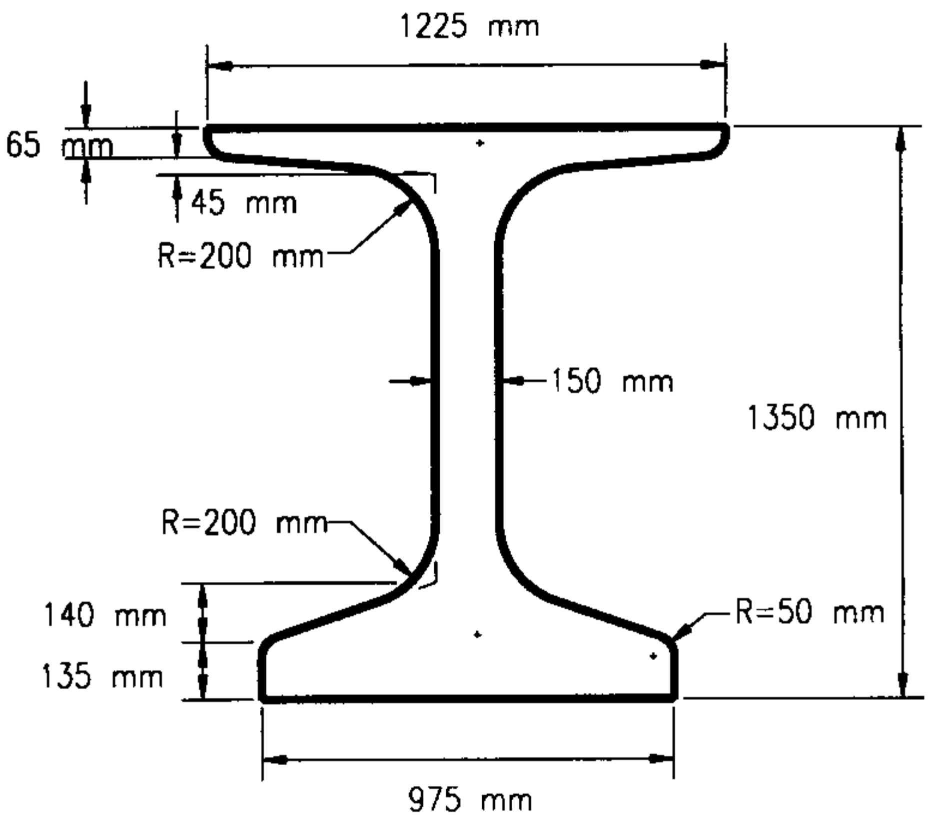


Fig. E7.4-2 Precast pretensioned NU 1350 girder. A_g = 486 051 mm^2.

Minimum Depth of Girder [A2.5.2.6.3]

For Prestressed
Precast I-Beams

$$h_{min} = 0.045\,L$$

$$= 0.045 \times 30{,}480 \text{ mm}$$

$$= 1{,}372 \text{ mm}$$

$$h_{actual} = \underbrace{1350}_{\text{Precast Girder Depth}} + \underbrace{190}_{\text{Slab Structural Depth}} \text{ mm}$$

$$= 1562 > 1372 \text{ mm} \quad \text{OK}$$

Effective Flange Widths [A 4.6.2.6.1]

Effective Span Length = L = 30,480 mm.

Interior Girders

$$b_i \le \begin{cases} \frac{1}{4} \text{ Effective Span} = \frac{1}{4} \times 30{,}480 = 7620 \text{ mm.} \\ 12\,t_s + \frac{1}{2}\,b_f = 12(190) + \frac{1}{2}(1225) = 2893 \\ \text{c/c spacing} = 2440 \text{ mm} = \underline{2440 \text{ mm}} \text{ Gover} \end{cases}$$

Example 7.10.4 Barker & Puckett

Effective Flange Widths

Exterior Girders [A 4.6.2.6.1]

$$be - \frac{bi}{2} \le \begin{cases} \frac{1}{8} \text{ Effective Span} & = \frac{1}{8} \times 30{,}480 = 3810 \\ 6ts + \frac{1}{4} bf & = 6 \times 190 + \frac{1}{4} \times 1225 = 2365 \\ \text{width of Overhang} & = 990 \text{ mm} = 990 \text{ mm} \end{cases}$$

Governs

$$\Rightarrow be = 990\text{ mm} + \frac{bi}{2} = 990 + \frac{2440}{2} = \underline{2210\text{ mm}}$$

Design of Conventional Reinforced Concrete Deck [A 4.6.2.1.6]

Critical Section for negative moment in deck slab is at 1/3 rd the Flange width, but not more than 380 mm from ℄ of the support for precast beams.

$$\frac{1}{3}\text{rd Flange Width} = \begin{cases} \frac{1}{3} \times 1225 & = 408 \text{ mm} \\ \text{or } 380 \text{ mm} & = \underline{380 \text{ mm governs}} \end{cases}$$

∴ Critical Section For Negative Bending may be taken at 380 mm from ℄ of support.

Rem: In Example 7.10.1 in the design of Top Steel for deck, critical section was at 175 mm from ℄ of Support.

** However, if critical section is taken at 380 mm from the support the negative steel requirement is reduced to <u>#10 Bars @ 250 mm c/c</u>

** The Positive Steel requirement remains unaffected at <u>#10 Bars @ 225 mm c/c</u>

Example 7.10.4 Barker & Puckett

Select Resistance Factors [A 5.5.4.2]

Strength Limit State	ϕ
– Flexure & Tension	1.0
– Shear & Torsion	0.9
– Compression in anchorage zones	0.8

Non Strength Limit States $\phi = 0.9$ [A 1.3.2.1]

Select Load Modifiers [A 1.3.2.1]

	Strength	Service	Fatigue	
Ductility, η_D	0.95	1.0	1.0	[A 1.3.3]
Redundancy, η_R	0.95	1.0	1.0	[A 1.3.4]
Importance, η_I	1.05	1.0	1.0	[A 1.3.5]
$\eta = \eta_D \eta_R \eta_I$	0.95	1.0	1.0	

Select Applicable Load Combinations A 3.

Strength - I Limit State

$$U = \eta \left[1.25 DC + 1.5 DW + 1.75 (LL + IM) + 1.0 FR + \gamma_{TG} TG \right]$$

Service Limit State - I

$$U = 1.0 (DC + DW) + 1.0 (LL + IM) + 0.3 (WS + WL) + 1.0 FR$$

Fatigue Limit State

$$U = 0.75 (LL + IM)$$

Service Limit State - III

$$U = 1.0 (DC + DW) + 0.8 (LL + IM) + 1.0 WA + 1.0 FR$$

Example 10.7.4 Barker & Puckett

Calculate Live Load Force Effects.

Select No. of Lanes [A 3.6.1.1.1]

$$NL = INT\left(\frac{w}{3600}\right) = INT\left(\frac{13420}{3600}\right)$$

$$= INT\,(3.7) \qquad = 3 \text{ Lanes.}$$

Multiple Presence Factor: [A 3.6.1.1.2]

No. of Loaded Lanes	m
1	1.2
2	1.0
3	0.85

Dynamic Load Allowance [A 3.6.2.1]

Components	IM(%)
- Deck Joints	75
- Fatigue & Fracture Limit State	15
- All other States	33

Example 10.7.4 Barker & Puckett

Distribution Factors for Moment: [A 4.6.2.2]

Cross-Section Type (K) [Table A 4.6.2.2.1-1]

Beam 55 MPa Concrete

Deck 30 MPa Concrete.

$$E_{concrete} = 0.043\ K_1\ \gamma_c^{1.5}\ \sqrt{f_c'} \qquad [A\ 5.4.2.4]$$

K_1 = Factor for Aggregate Source

γ_c = Unit density of Concrete

f_c' = Concrete compressive strength.

$$\Rightarrow \text{Modular Ratio} = n = \frac{E_B}{E_D} = \frac{\sqrt{f_c'}\,(\text{Beam})}{\sqrt{f_c'}\,(\text{Deck})}$$

$$n = \sqrt{\frac{55}{30}} = 1.354$$

$$\text{Longitudinal Stiffness Parameter} = K_g = n\left(I + A e_g^2\right)$$

$$e_g = 742 + 50 + \frac{190}{2} = 887\ mm$$

$$A_g = 486{,}051\ mm^2$$

$$I_g = 126{,}011\ mm^4$$

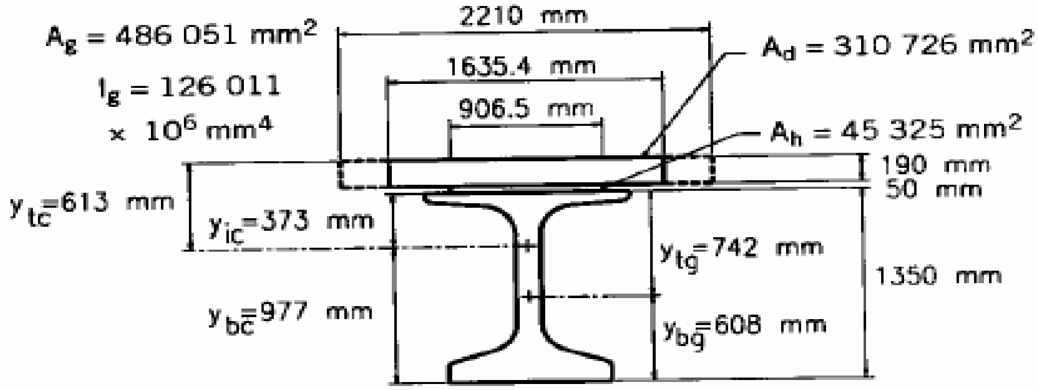


Fig. E7.4-8 Composite section properties.

Example 10.7.4 Barker & Puckett

$$K_g = n\left(I_g + A e_g^2\right)$$
$$= 1.354\left(126{,}011 + 486{,}051(887)^2\right) = 517.8 \times 10^9 \text{ mm}^4$$

$$\frac{K_g}{L t_s^3} = \frac{517.8 \times 10^9}{30480 \times (190)^3} = 2.477$$

L = c/c Span between Bearings of the Bridge = 30,480 mm

S = c/c Spacing between girders = 2440 mm

Determination of Girder Moment/Shear Coefficients

One Design Lane Loaded

$$mg_M^{SI} = 0.06 + \left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{L t_s^3}\right)^{0.1} \quad [A\ 4.6.2.2b\text{-}1]$$

$$= 0.06 + \left(\frac{2440}{4300}\right)^{0.4}\left(\frac{2440}{30480}\right)^{0.3}\left(2.477\right)^{0.1} = 0.469$$

Two Design Lanes Loaded

$$mg_M^{MI} = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{L t_s^3}\right)^{0.1} \quad [A\ 4.6.2.2d\text{-}1]$$

$$= 0.075 + \left(\frac{2440}{2900}\right)^{0.6}\left(\frac{2440}{30480}\right)^{0.2}\left(2.477\right)^{0.1} = 0.671$$

Governs

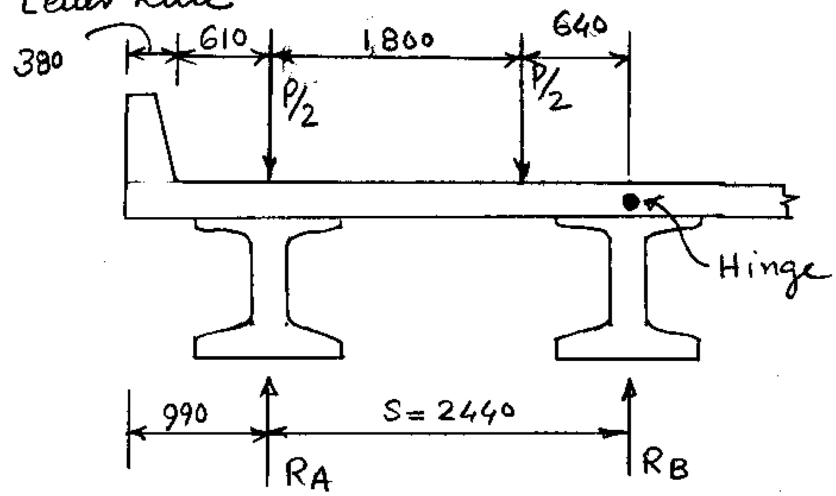
Example 10.7.4 Barker & Puckett

Girder Moment/Shear Coefficients

Exterior Beams

One Design Lane Loaded [A4.6.2.2.2d]

Using Lever Rule



$$RA = \frac{P}{2} + \frac{P}{2} \times \frac{640}{2440} = 0.631\,P$$

$$mg_M^{SE} = 1.2 \times 0.631 = 0.757 \simeq 0.762$$

Used in Text.

Two or More Design Lanes Loaded

[A4.6.2.2.2d-1]

d_e = Dist. from exterior beam to interior edge of the Curb

$$= 990 - 380 = 610 \text{ mm}$$

$$mg_M^{ME} = e \cdot mg_M^{MI}$$

$$e = 0.77 + \frac{d_e}{2800} \geq 1.0 = 0.77 + \frac{610}{2800} = 0.988 < 1.0$$

Use 1.0

$$mg_M^{ME} = 1.0 \times mg_M^{MI} = 1.0 \times 0.671 = 0.671$$

Example 10.7.4 Barker & Puckett

Distribution Factors for Shear

Interior Beams

One Design Lane Loaded [A 4.6.2.2.3a-1]

$$mg_V^{SI} = 0.36 + \frac{S}{7600} = 0.36 + \frac{2440}{7600} = 0.68$$

Two Design Lanes Loaded

$$mg_V^{MI} = 0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^{2.0}$$

$$= 0.2 + \frac{2440}{3600} - \left(\frac{2440}{10700}\right)^{2.0} = 0.826$$

Governs

Exterior Beams [A 4.6.2.2.3b-1]

One Design Lane Loaded –

Use Lever Rule

$$mg_V^{SE} = \qquad = 0.762$$

Governs

Two Design Lanes Loaded

$$e = 0.6 + \frac{d_e}{3000} = 0.6 + \frac{610}{3000} = 0.803$$

$$mg_V^{ME} = e \cdot mg_V^{MI} = 0.803 \times 0.826 = 0.663$$

6. *Calculation of shears and moments due to live loads* The shears and moments at tenth points along the span are found next. Calculations are shown below for Locations 100, 101, and 105 only. Concentrated loads are multiplied by influence line ordinates. Uniform loads are multiplied by the area under the influence line. As discussed in Chapter 5, the influence functions are straight lines for simple spans. Shears and moments at the other locations are found in a similar manner. Results of these calculations are summarized in Tables E7.4-3 and E7.4-4.

Location 100 (Fig. E7.4-4)

Truck

$$V_{100}^{Tr} = 145\left(1 + \frac{26\,180}{30\,480}\right) + 35\left(\frac{21\,880}{30\,480}\right) = 294.7 \text{ kN}$$

$$M_{100}^{Tr} = 0$$

Lane

$$V_{100}^{Ln} = \frac{(15\,240)(9.3)}{10^3} = 141.7 \text{ kN}$$

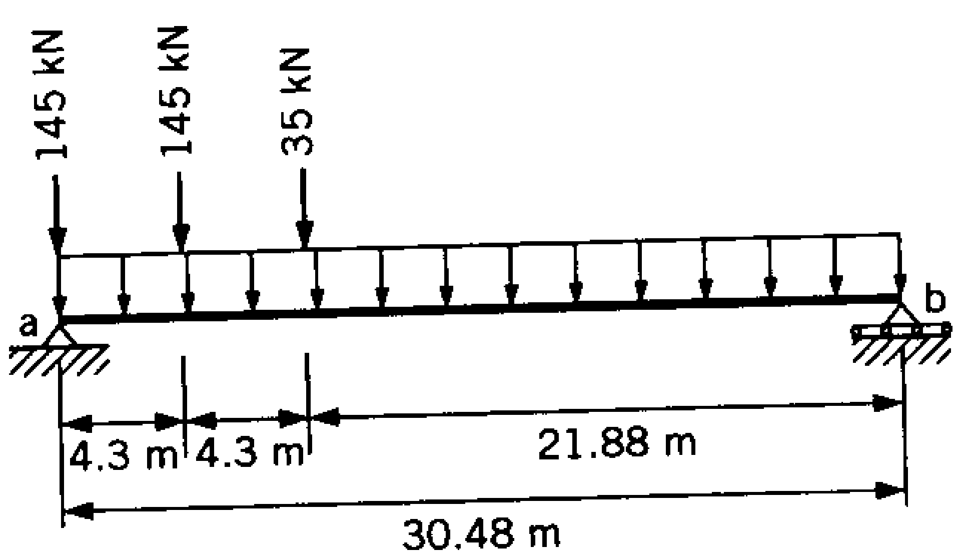


Fig. E7.4-4 Live load placement at Location 100.

$$M_{100}^{Ln} = 0$$

Location 101 (Fig. E7.4-5):

Truck

$$V_{101}^{Tr} = R_a = 145\left(\frac{23\,132 + 27\,432}{30\,480}\right) + 35\left(\frac{18\,832}{30\,480}\right) = 262.2 \text{ kN}$$

$$M_{101}^{Tr} = \frac{262.2(3048)}{10^3} = 799.2 \text{ kN m}$$

Tandem

$$V_{101}^{Ta} = 110\left(\frac{26\,232 + 27\,432}{30\,480}\right) = 193.7 \text{ kN}$$

$$M_{101}^{Ta} = \frac{193.7(3048)}{10^3} = 590.4 \text{ kN m}$$

Lane

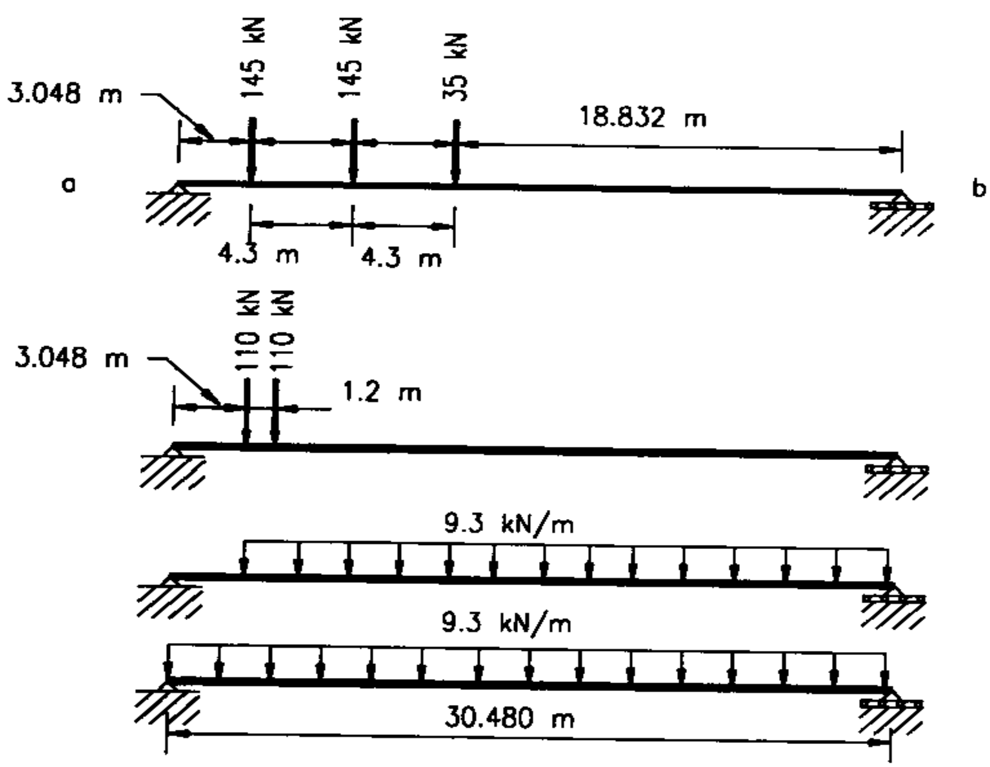


Fig. E7.4-5 Live load placement at Location 101.

$$V_{101}^{Ln} = \frac{9.3(27\,432)}{10^3}\left(\frac{13\,716}{30\,480}\right) = 114.8 \text{ kN}$$

$$M_{101}^{Ln} = \frac{1}{2}wab = \frac{1}{2}\frac{(9.3)(3048)(27\,432)}{10^6} = 388.8 \text{ kN m}$$

Location 105 (Fig. E7.4-6):

Truck

$$V_{105}^{Tr} = 145\left(\frac{15\,240 + 10\,940}{30\,480}\right) + 35\left(\frac{6640}{30\,480}\right) = 132.2 \text{ kN}$$

$$R_a = 145\left(\frac{15\,240 + 10\,940}{30\,480}\right) + 35\left(\frac{19\,540}{30\,480}\right) = 147 \text{ kN}$$

$$M_{105}^{Tr} = \frac{(147)(15\,240) - 35(4300)}{10^3} = 2090 \text{ kN m}$$

Tandem

$$V_{105}^{Ta} = 110\left(\frac{15\,240 + 14\,040}{30\,480}\right) = 105.7 \text{ kN}$$

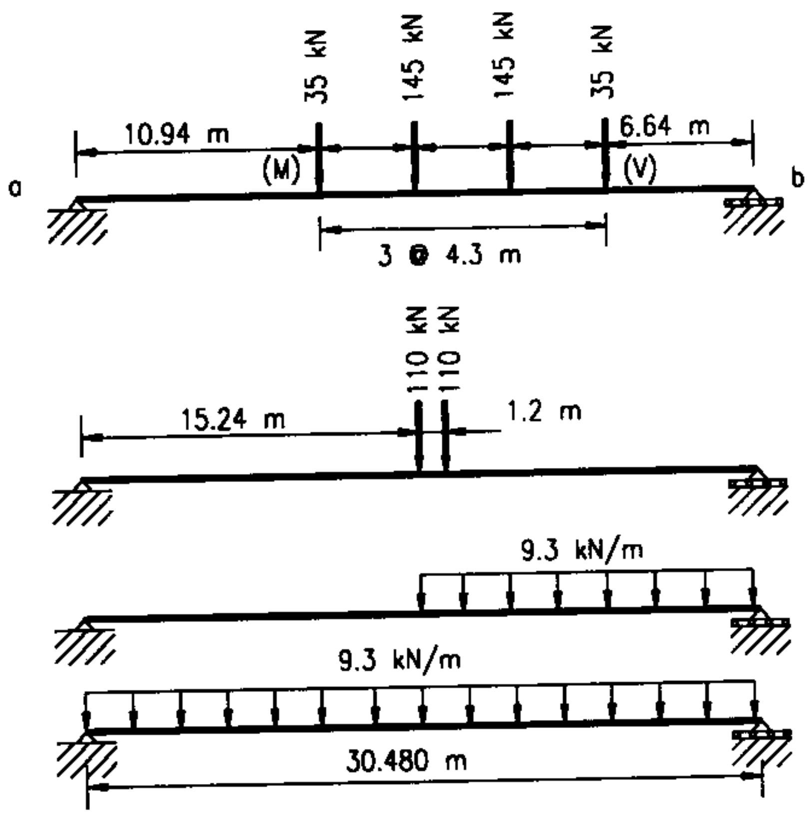


Fig. E7.4-6 Live load placement at Location 105.

$$M_{105}^{Ta} = \frac{(105.7)(15\,240)}{10^3} = 1610 \text{ kN m}$$

Lane

$$V_{105}^{Ln} = \frac{9.3(15\,240)}{10^3}\left(\frac{7620}{30\,480}\right) = 35.4 \text{ kN}$$

$$M_{105}^{Ln} = \frac{1}{8}wL^2 = \frac{1}{8}\,\frac{(9.3)(30\,480)^2}{10^6} = 1080 \text{ kN m}$$

H. ***Calculate Force Effects from Other Loads***

1. *Interior girders*

DC Weight of concrete = $(2400)(9.81)(10^{-9}) = 2.3544 \times 10^{-5}$ N/mm³

$$\begin{aligned}
\text{Slab}\quad & (2.3544 \times 10^{-5})(205)(2440) = 11.78 \text{ N/mm} \\
\text{50-mm haunch}\quad & (2.3544 \times 10^{-5})(50)(1225) = 1.44 \text{ N/mm} \\
\text{Girder}\quad & (2.3544 \times 10^{-5})(486\,051) = \underline{11.44 \text{ N/mm}} \\
& = 24.66 \text{ N/mm}
\end{aligned}$$

Estimate diaphragm size 300 mm thick, 1200 mm deep

Diaphragms @ 1/3 points $(2.3544 \times 10^{-5})(300)(1200)(2290)$

$= 19\,410$ N

DW 75 mm bituminous paving = $(2250)(9.81)(10^{-9})(75)(2440)$

$= 4.04$ N/mm

2. *Exterior girders*

$$\begin{aligned}
DC1\text{: Overhang}\quad & (5.42 \times 10^{-3})(990) = 5.36 \text{ N/mm} \\
\text{Slab}\quad & (4.83 \times 10^{-3})(1220) = 5.89 \text{ N/mm} \\
\text{Girder + Haunch}\quad & = \underline{12.88 \text{ N/mm}} \\
& = 24.13 \text{ N/mm}
\end{aligned}$$

Diaphragms @ $\frac{1}{3}$ points $19\,410/2 = 9705\ N = 9.705$ kN

$DC2$: Barrier = 4.65 N/mm

DW 75-mm bituminous paving

$= (1.66 \times 10^{-3})(990 - 380 + 1220)$

$= 3.04$ N/mm

($DC2$ and DW act on the composite section)

From Figure E7.4-7, shears and moments due to a unit uniform load are found at tenth points (Table E7.4-1), where

$$V_x = w\left(\frac{L}{2} - x\right) = wL(0.5 - \xi),\ \xi = \frac{x}{L}$$

$$M_x = \frac{w}{2}x(L - x) = 0.5wL^2(\xi - \xi^2)$$

From Figure E7.4-7, shears and moments due to the diaphragms for interior girders are found at tenth points (Table E7.4-2). Values for exterior girders are one-half the values for interior girders.

3. *Summary of force effects*
   a. Interior girders (Table E7.4-3)

$$mg_M = 0.671,\ mg_V = 0.826,\ IM^{TR} = 33\%,\ IM^{LN} = 0,$$

$w_g = 11.44$ N/mm (Weight of Girder)

(Total Weight on Interior Girder) $DC1 = 24.66$ N/mm, Diaphragm = 19 410 N,

$DW = 4.04$ N/mm (Weight Wearing Course)

   b. Exterior girders (Table E7.4-4)

$$mg_M = 0.762,\ mg_V = 0.762,\ IM^{TR} = 33\%,\ IM^{LN} = 0$$

$$DC1 = 24.13 \text{ N/mm, Diaphragm} = 9705 \text{ N},$$

$$DC2 = 4.65 \text{ N/mm},\ DW = 3.04 \text{ N/mm}$$

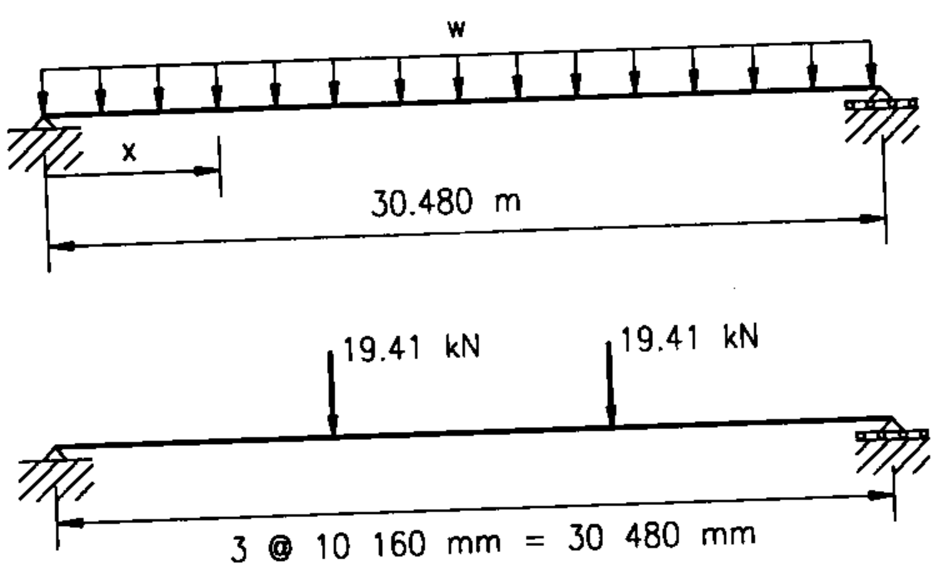


Fig. E7.4-7 Uniform dead and diaphragm loads.

TABLE E7.4-1 Shears and Moments for w = 1.0 N/mm = 1.0 kN/m

	$\xi = 0$	$\xi = 0.1$	$\xi = 0.2$	$\xi = 0.3$	$\xi = 0.4$	$\xi = 0.5$
V_x (kN)	15.240	12.192	9.144	6.096	3.048	0
M_x (kN m)	0	41.8	74.3	97.5	111.5	116.1

I. *Investigate Service Limit State*

1. *Stress limits for prestressing tendons:* (Table 7.8) [A5.9.3]

f_{pu} = 1860 MPa, low-relaxation 12.70 mm, seven-wire strands
$A = 98.71$ mm^2 (Table B.2), E_p = 197 000 MPa [A5.4.4.2]

Pretensioning

At jacking $\quad f_{pj} = 0.78 f_{pu} = 0.78(1860) = 1451$ MPa

After transfer $\quad f_{pt} = 0.74 f_{pu} = 0.74(1860) = 1376$ MPa

$f_{py} = 0.9 f_{pu} = 0.9(1860) = 1674$ MPa

After losses $\quad f_{pe} = 0.80 f_{py} = 0.80(1674) = 1339$ MPa

2. *Stress limits for concrete:* (Tables 7.6 and 7.7) [A5.9.4]

f'_c = 55 MPa, 28-day compressive strength
$f'_{ci} = 0.75 f'_c$ = 40 MPa strength at time of initial prestressing

Temporary stresses before losses—fully prestressed components:

Compressive stresses $\quad f_{ci} = 0.6 f'_{ci} = 0.6(40) = 24$ MPa

Tensile stresses $\quad f_{ti} = 0.25\sqrt{f'_{ci}} = 0.25\sqrt{40} = 1.58$ MPa

$1.58 > 1.38$ (Use $f_{ti} = 1.38$ MPa)

TABLE E7.4-2 Shears and Moments due to Diaphragm, Interior Girders

	$\xi = 0$	$\xi = 0.1$	$\xi = 0.2$	$\xi = 0.3$	$\xi = 0.4$	$\xi = 0.5$
V_x (kN)	19.410	19.410	19.410	19.410	0	0
M_x (kN m)	0	59.2	118.3	177.5	197.2	197.2

TABLE E7.4-3 Summary of Force Effects for Interior Girder

Force Effect	Load Type	Distance from Support 0	0.1L	0.2L	0.3L	0.4L	0.5L
	Service *I* Loads						
M_s (kN m)	Girder self-weight	0	478	850	1115	1276	1328
	$DC1$ (incl. diaph.) on girder alone	0	1090	1951	2582	2947	3060
	DW on composite section	0	169	300	394	451	469
	$mg_M(LL + IM)$	0	974	1714	2216	2518	2590
V_s (kN)	$DC1$ (incl. diaph.) on girder alone	395	320	245	170	75	0
	DW on composite section	62	49	37	25	12	0
	$mg_V(LL + IM)$	441	383	327	274	223	175
	Strength *I* Loads						
M_u (kN m)	$\eta[1.25DC + 1.50DW + 1.75(LL + IM)]$	0	3154	5594	7312	8328	8608
V_u (kN)	$\eta[1.25DC + 1.50DW + 1.75(LL + IM)]$	1291	1087	887	693	477	290

TABLE E7.4-4 Summary of Force Effects for Exterior Girder

Force Effect	Load Type	Distance from Support					
		0	0.1*L*	0.2*L*	0.3*L*	0.4*L*	0.5*L*
	Service *I* Loads						
M_s (kN m)	Girder self-weight	0	478	850	1115	1276	1328
	*DC*1 (incl. diaph.) on girder alone	0	1038	1852	2441	2789	2900
	*DC*2 (barrier) on composite section	0	194	346	453	519	540
	DW on composite section	0	127	226	296	339	353
	$mg_M(LL + IM)$	0	1106	1946	2516	2860	2941
V_s (kN)	*DC*1 (incl. diaph.) on girder alone	377	304	230	157	74	0
	*DC*2 (barrier) on composite section	71	57	43	28	14	0
	DW on composite section	46	37	28	19	9	0
	$mg_V(LL + IM)$	407	353	302	253	206	161
	Strength *I* Loads						
M_u (kN m)	$\eta[1.25DC + 1.50DW + 1.75(LL + IM)]$	0	3483	6167	8041	9166	9477
V_u (kN)	$\eta[1.25DC + 1.50DW + 1.75(LL + IM)]$	1274	1068	866	667	460	268

Stresses at service limit state after losses—fully prestressed components [A5.9.4.2]:

Compressive stresses $f_c = 0.45f'_c$

$$= 0.45(55) = 24.75 \text{ MPa}$$

Tensile stresses $f_t = 0.5\sqrt{f'_c}$

$$= 0.5\sqrt{55}$$

$$= 3.71 \text{ MPa} \quad \text{Use Service } III$$

Modulus of Elasticity

$$E_{ci} = 4800\sqrt{f'_{ci}} = 4800\sqrt{40} = 30\ 360 \text{ MPa}$$

$$E_c = 4800\sqrt{f'_c} = 4800\sqrt{55} = 35\ 600 \text{ MPa}$$

3. *Preliminary choices of prestressing tendons* Controlled either by the concrete stress limits at service loads or by the sectional strength under factored loads. For the final load condition, the composite cross section properties are needed. To transform the CIP deck into equivalent girder concrete, the modular ratio is taken as $n = \sqrt{30/55} = 0.74$.

... depth is 50 mm

If we assume for convenience that the haunch depth is 50 mm and use the effective flange width of 2210 mm for an exterior girder, the composite section dimensions are shown in Figure E7.4-8.

Section properties for the girder are as follows (Green and Tadros, 1994):

$$A_g = 486\ 051\ \text{mm}^2$$

$$I_g = 126\ 011 \times 10^6\ \text{mm}^4$$

$$S_{tg} = \frac{I_g}{y_{tg}} = \frac{126.0 \times 10^9}{742} = 169.8 \times 10^6\ \text{mm}^3$$

$$S_{bg} = \frac{I_g}{y_{bg}} = \frac{126.0 \times 10^9}{608} = 207.3 \times 10^6\ \text{mm}^3$$

Section properties for the composite girder are calculated below.

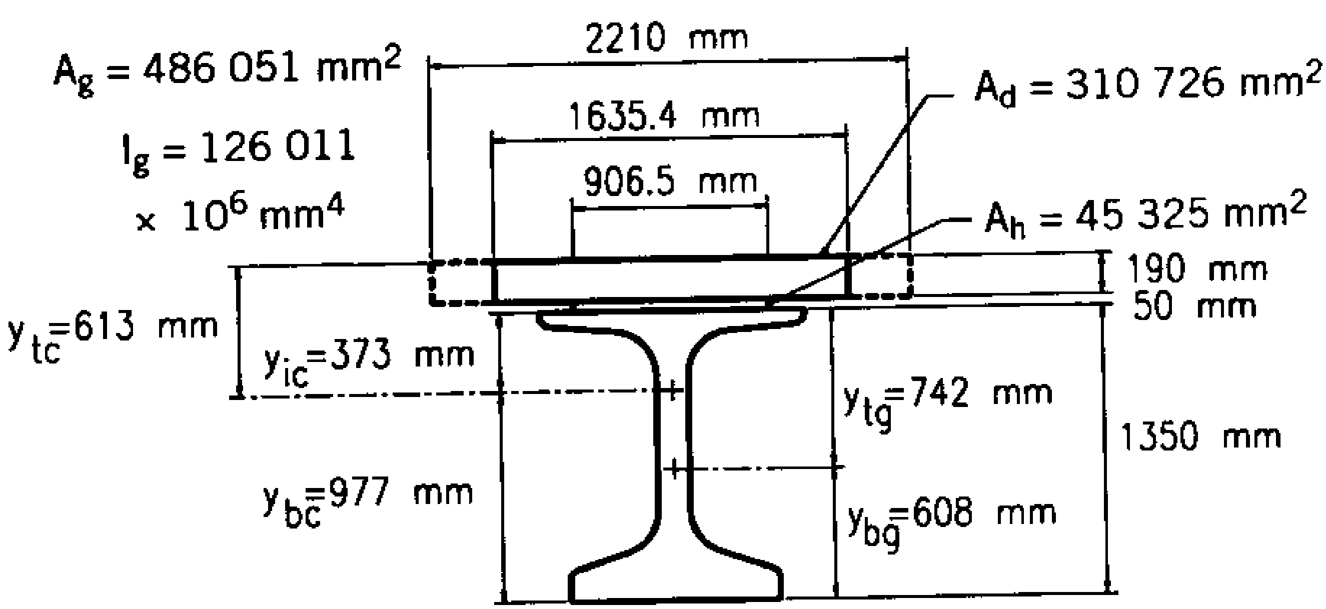


Fig. E7.4-8 Composite section properties.

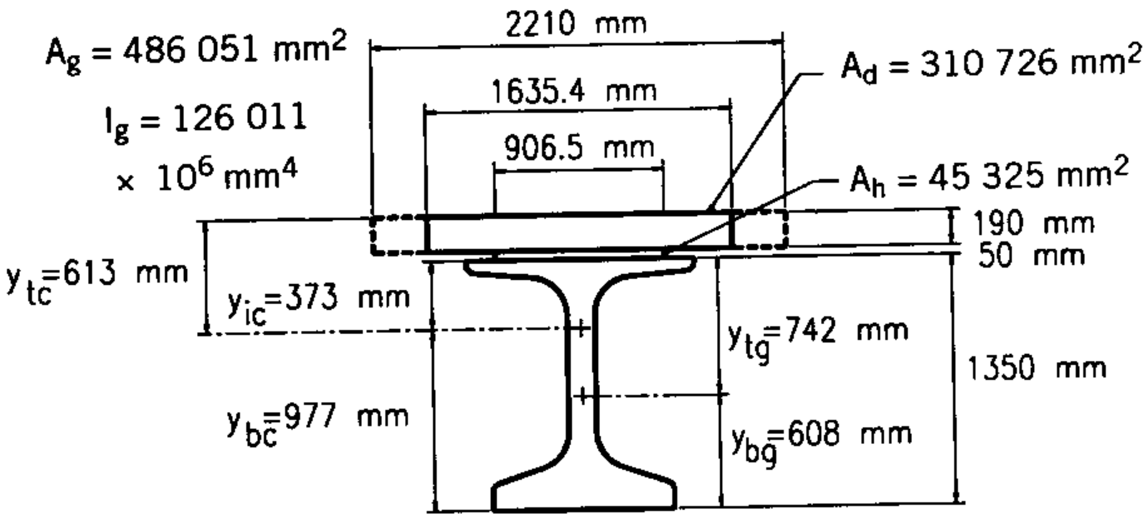


Fig. E7.4-8 Composite section properties.

The distance to the neutral axis from the top of the deck is

$$\bar{y} = \frac{(310\ 726)(95) + (45\ 325)(215) + (486\ 051)(982)}{310\ 726 + 45\ 325 + 486\ 051} = 613 \text{ mm}$$

$$I_c = (126\ 011 \times 10^6) + (486\ 051)(369)^2 + (906.5)(50)^3/12$$
$$+ (45\ 325)(398)^2 + (1635.4)(190)^3/12 + (310\ 726)(518)^2$$
$$= 283.7 \times 10^9 \text{ mm}^4$$

$$S_{tc} = \frac{I_c}{y_{tc}} = \frac{283.7 \times 10^9}{613} = 462.8 \times 10^6 \text{ mm}^3 \text{ (top of deck)}$$

$$S_{ic} = \frac{I_c}{y_{ic}} = \frac{283.7 \times 10^9}{373} = 760.6 \times 10^6 \text{ mm}^3 \text{ (top of girder)}$$

$$S_{bc} = \frac{I_c}{y_{bc}} = \frac{283.7 \times 10^9}{977} = 290.4 \times 10^6 \text{ mm}^3 \text{ (bot of girder)}$$

Preliminary Analysis—Exterior Girder at Midspan

The minimum value of prestress force F_f to ensure that the tension in the bottom fiber of the beam does not exceed the limit of 3.71 MPa in the composite section under final conditions can be expressed as (Eq. 7.53)

$$f_{bg} = -\frac{F_f}{A_g} - \frac{F_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_L}{S_{bc}} \leq 3.71 \text{ MPa}$$

where

M_{dg} = moment due to self-weight of girder = 1328 kN m

M_{ds} = moment due to dead load of wet concrete + diaphragm

= 1572 kN m (= 2900 – 1328)

M_{da} = moment due to additional dead load after concrete hardens

= 893 kN m (= 540 + 353)

M_L = moment due to live load

+ impact (Service *III*) = 0.8(2941) (Service III Limit State)

= 2353 kN m

e_g = distance from *cg* of girder to centroid of pretensioned strands

= 608 − 100 = 508 mm

$$f_{bg} = -\frac{F_f}{486\,051} - \frac{F_f(508)}{207.3 \times 10^6} + \frac{2900 \times 10^6}{207.3 \times 10^6} + \frac{3246 \times 10^6}{290.4 \times 10^6} \le 3.71 \text{ MPa}$$

$$= -\left[(2.057 \times 10^{-6}) + (2.451 \times 10^{-6})\right]F_f + 13.989 + 11.118 \le 3.71$$

$$(4.508 \times 10^{-6})F_f \ge 21.397$$

$$F_f \ge \frac{21.397}{4.508 \times 10^{-6}} = 4.75 \times 10^6 \text{ N} = 4750 \text{ kN}$$

Assuming stress in strands after all losses is $0.6f_{pu} = 0.6(1860) = 1116$ MPa = 1116 N/mm²,

$$A_{ps} \ge \frac{F_f}{0.6f_{pu}} = \frac{4.75 \times 10^6}{1116} = 4250 \text{ mm}^2$$

From Collins and Mitchell (1991), in order to satisfy strength requirements, the following approximate expression can be used

$$\phi M_n = \phi(A_{ps} \times 0.95f_{pu} + A_s f_y)\, 0.9h \ge M_u$$

where

$$\phi = 1.0$$

$$PPR = 1.0 \text{ (prestress ratio) [A5.5.4.2.1]}$$

$$h = \text{overall depth of composite section} = 1590 \text{ mm}$$

$$M_u = \text{Strength } I \text{ factored moment} = 9477 \text{ kN m}$$

$$A_{ps} \geq \frac{M_u}{\phi 0.95 f_{pu}(0.9h)} = \frac{9477 \times 10^6}{1.0(0.95)(1860)(0.9)(1590)}$$

$$A_{ps} \geq 3748 \text{ mm}^2 < 4250 \text{ mm}^2, \text{ strength limit is not critical}$$

Try 52—12.70-mm strands $A_{ps} = 52(98.71) = 5133$ mm² (Fig. E7.4-9)

(*Note:* Other strand patterns were tried. To save space, only the final iteration is given here.)

At Midspan			At End Section		
N	y	Ny	N	y	Ny
32	75	2400	28	75	2 100
10	150	1500	8	150	1 200
6	200	1200	4	200	800
4	275	1100	12	1175	14 100
52		6200	52		18 200

$$\bar{y} = \frac{6200}{52} = 119 \text{ mm} \qquad \bar{y} = \frac{18\,200}{52} = 350 \text{ mm}$$

$$e_{CL} = 608 - 119 = 489 \text{ mm} \qquad e_{end} = 608 - 350 = 258 \text{ mm}$$

4. *Evaluate prestress losses* [A5.9.5]

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR}$$

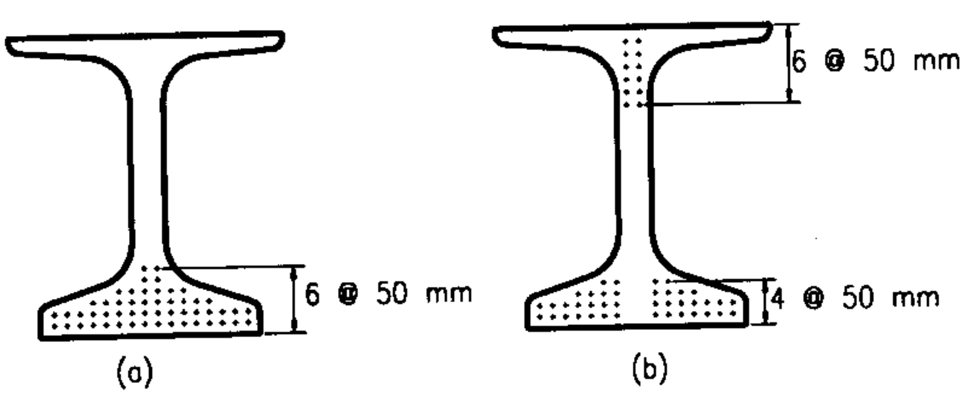


Fig. E7.4-9 Strand patterns at (a) midspan and (b) support.

a. Elastic shortening, Δf_{pES} (Eq. 7.96) [A5.9.5.2.3a]

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

where

$$E_p = 197\ 000 \text{ MPa}$$

$$E_{ci} = 4800\sqrt{40} = 30\ 360 \text{ MPa}$$

f_{cgp} = sum of concrete stresses at cg of A_{ps} due to F_i and M_{dg} at centerline

$$F_i = 0.70 f_{pu} A_{ps} = 0.70(1860)(52)(98.71) \times 10^{-3}$$

$$F_i = 6683 \text{ kN}$$

$$M_{dg} = 1328 \text{ kN m}$$

$$f_{cgp} = -\frac{F_i}{A_g} - \frac{F_i e_{CL}^2}{I_g} + \frac{M_{dg} e_{CL}}{I_g}, \qquad CL = \text{centerline}$$

$$= -\frac{6.683 \times 10^6}{486\ 051} - \frac{(6.683 \times 10^6)(489)^2}{126\ 011 \times 10^6} + \frac{(1328 \times 10^6)(489)}{126\ 011 \times 10^6}$$

$f_{cgp} = -21.3$ MPa (Minus sign indicates elastic shortening of concrete. This results in a positive prestress loss.)

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} = \frac{197\ 000}{30\ 360}(21.3) = 138.1 \text{ MPa}$$

Relaxation, Δf_{pR1} (Eq. 7.115) [A5.9.5.4.4]

$$\Delta f_{pR} = \Delta f_{pR1} + \Delta f_{pR2}$$

where

$$\Delta f_{pR1} = \text{Relaxation loss at transfer (Eq. 7.116)}$$

$$= \frac{\log(24t)}{40.0}\left(\frac{f_{pi}}{f_{py}} - 0.55\right) f_{pi}$$

t = Estimated time from prestressing to transfer

= 4 days (max)

f_{pi} = Initial stress in tendon at the end of stressing

1st Iteration (Eq. 7.117)

$$f_{pi} = f_{pt} - \Delta f_{pES} = 0.74(1860) - 138.1 = 1238.3 \text{ MPa}$$

$$f_{py} = 0.9 f_{pu} = 0.9(1860) = 1674 \text{ MPa}$$

$$\Delta f_{pR1} = \frac{\log[24(4)]}{40}\left(\frac{1238.3}{1674} - 0.55\right)1238.3 = 11.6 \text{ MPa}$$

Recalculating f_{pi} and Δf_{pR1}

$$f_{pi} = 1238.3 - 11.6 = 1226.7 \text{ MPa}$$

$$\Delta f_{pR1} = \frac{\log[24(4)]}{40}\left(\frac{1226.7}{1674} - 0.55\right)1226.7 = 11.1 \text{ MPa}$$

2nd Iteration (Eq. 7.117)

$$f_{pi} = 0.74(1860) - (138.1 + 11.1) = 1227.2 \text{ MPa}$$

$$F_i = (52)(98.71)(1.2272) = 6299.1 \text{ kN}$$

$$f_{cgp} = -\frac{6.2291 \times 10^6}{486\ 051} - \frac{(6.2991 \times 10^6)(489)^2}{126\ 011 \times 10^6}$$

$$+ \frac{(1328.2 \times 10^6)(489)}{126\ 011 \times 10^6} = -19.8 \text{ MPa}$$

$$\Delta f_{pES} = \frac{197\ 000}{30\ 360}(19.8) = 128.5 \text{ MPa}$$

$$f_{pi} = 0.74(1860) - (128.5 + 11.1) = 1236.8 \text{ MPa}$$

$$\Delta f_{pR1} = \frac{\log[24(4)]}{40}\left(\frac{1236.8}{1674} - 0.55\right)1236.8 = 11.57 \text{ MPa}$$

3rd Iteration

$$f_{pi} = 0.74(1860) - (128.5 + 11.57) = 1236.3 \text{ MPa}$$

$$F_i = (52)(98.71)(1.2363) = 6346 \text{ kN}$$

$$f_{cgp} = -\frac{6.346 \times 10^6}{486\ 051} - \frac{(6.346 \times 10^6)(489)^2}{126\ 011 \times 10^6} + \frac{(1328.2 \times 10^6)(489)}{126\ 011 \times 10^6} = -19.9 \text{ MPa}$$

$$\Delta f_{pES} = \frac{197\ 000}{30\ 360}(19.9) = 129.1 \text{ MPa}$$

$$f_{pi} = 0.74(1860) - (129.1 + 11.57) = 1235.7 \text{ MPa}, \qquad \text{OK}$$

b. Shrinkage, Δf_{pSR} (Eq. 7.105) [A5.9.5.4.2]

$$\Delta f_{pSR} = 117 - 1.03H$$

where

H = relative humidity = 70% for Virginia [Fig. 5.4.2.3.3-1]

$$\Delta f_{pSR} = 117 - 1.03(70) = 44.9 \text{ MPa}$$

c. Creep, Δf_{pCR} (Eq. 7.114) [A5.9.5.4.3]

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} \geq 0$$

where

f_{cgp} = concrete stress at cg of A_{ps} at transfer = 19.9 MPa

Δf_{cdp} = change in concrete stress at cg of A_{ps} due to permanent loads except the load acting when F_i is applied, that is subtract M_{dg}

$$\Delta f_{cdp} = -\frac{(2900 - 1328)10^6(489)}{126\ 011 \times 10^6} - \frac{(540 + 353)10^6(977 - 119)}{283.7 \times 10^9} = -8.8 \text{ MPa}$$

(Again, a negative concrete stress results in a positive prestress loss.)

$$\Delta f_{pCR} = 12.0(19.9) - 7(8.8) = 177.2 \text{ MPa}$$

d. Relaxation, Δf_{pR} (Eq. 7.115) [A5.9.5.4.4]

$$\Delta f_{pR} = \Delta f_{pR1} + \Delta f_{pR2}$$

where Δf_{pR1} = relaxation loss at transfer = 11.57 MPa

$$\begin{aligned}\Delta f_{pR2} &= 0.3[138 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \\ &= 0.3[138 - 0.4(129.1) - 0.2(44.9 + 177.2)] \\ \Delta f_{pR2} &= 12.6 \text{ MPa}\end{aligned}$$

e. Total losses (Eq. 7.93)

$$\begin{aligned}\Delta f_{pT} &= \text{(initial losses)} + \text{(long-term losses)} \\ &= (\Delta f_{pES} + \Delta f_{pR1}) + (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}) \\ &= (129.1 + 11.57) + (44.9 + 177.2 + 12.6) \\ \Delta f_{pT} &= 375.4 \text{ MPa}\end{aligned}$$

Approximate Lump Sum Estimate of Time-Dependent Losses

Time-dependent losses can be approximated by the following formula, given in Table 7.12 [Table A5.9.5.3-1]

$$\text{Average} = 230\left[1 - 0.15\frac{f'_c - 41}{41}\right] + 41PPR$$

where

$$PPR = 1.0 \quad \text{[A5.5.4.2.1]}$$

$$\text{Average} = 230\left[1 - 0.15\,\frac{55 - 41}{41}\right] + 41 = 259.2 \text{ MPa}$$

For low relaxation strand,

$$\text{Average} = 259.2 - 41 = 218.2 \text{ MPa}$$

Δf_{pT} = 129.1 + 218.2 = 347.3 MPa, compared to the losses of 375.4 MPa, calculated by the longer procedure. (This amounts to a underestimation of total stresses of 7.5%.)

5. *Calculate Girder Stresses at Transfer*

$$F_i = 6346 \text{ kN}$$

$$e_{CL} = 489 \text{ mm} \qquad e_{\text{end}} = 238 \text{ mm}$$

At Midspan

$$\begin{aligned} f_{ti} &= -\frac{F_i}{A_g} + \frac{F_i e_{CL}}{S_{tg}} - \frac{M_{dg}}{S_{tg}} \\ &= -\frac{6.346 \times 10^6}{486\ 051} + \frac{(6.346 \times 10^6)(489)}{169.8 \times 10^6} - \frac{1328 \times 10^6}{169.8 \times 10^6} \\ &= -2.5 \text{ MPa} < 1.38 \text{ MPa}, \qquad \text{OK} \end{aligned}$$

$$\begin{aligned} f_{bi} &= -\frac{F_i}{A_g} - \frac{F_i e_{CL}}{S_{bg}} + \frac{M_{dg}}{S_{bg}} \\ &= -\frac{6.346 \times 10^6}{486\ 051} + \frac{(6.346 \times 10^6)(489)}{207.3 \times 10^6} - \frac{1328 \times 10^6}{207.3 \times 10^6} \\ &= -21.6 \text{ MPa} > f_{ci} = -24 \text{ MPa}, \qquad \text{OK} \end{aligned}$$

At Beam End

$$\begin{aligned} f_{ti} &= -\frac{6.346 \times 10^6}{486\ 051} - \frac{(6.346 \times 10^6)(258)}{169.8 \times 10^6} \\ &= -3.4 \text{ MPa} < 1.38 \text{ MPa}, \qquad \text{OK} \end{aligned}$$

$$\begin{aligned} f_{bi} &= -\frac{6.346 \times 10^6}{486\ 051} - \frac{(6.346 \times 10^6)(258)}{207.3 \times 10^6} \\ &= -21.0 \text{ MPa} > f_{ci} = -24 \text{ MPa}, \qquad \text{OK} \end{aligned}$$

6. *Girder Stresses After Total Losses*

$$f_{pf} = 0.74 f_{pu} - \Delta f_{pT} = 1376.4 - 375.4 = 1001.0 \text{ MPa}$$

$$F_f = \frac{(5133)(1001)}{10^3} = 5140 \text{ kN}$$

At Midspan

$$f_{tf} = -\frac{F_f}{A_g} + \frac{F_f e_{CL}}{S_{tg}} - \frac{M_{dg} + M_{ds}}{S_{tg}} - \frac{M_{da} + M_L}{S_{ic}}$$

$$= -\frac{5.14 \times 10^6}{486\ 051} + \frac{(5.14 \times 10^6)(489)}{169.8 \times 10^6} - \frac{(1328 + 1572) \times 10^6}{169.8 \times 10^6}$$

$$- \frac{(893 + 2941) \times 10^6}{760.6 \times 10^6}, \text{ Service I}$$

$$= -17.9 \text{ MPa} > f_c = 0.45 f'_c = -24.75 \text{ MPa}, \qquad \text{OK}$$

$$f_{bf} = -\frac{5.14 \times 10^6}{486\ 051} - \frac{(5.14 \times 10^6)(489)}{207.3 \times 10^6} + \frac{2900 \times 10^6}{207.3 \times 10^6}$$

$$+ \frac{(893 + 0.8 \times 2941) \times 10^6}{290.4 \times 10^6}, \text{ Service III}$$

$$= 2.44 \text{ MPa} < 3.71 \text{ MPa}, \qquad \text{OK}$$

52 – 12.70-mm low-relaxation strands satisfy Service Limit State

7. *Check Fatigue Limit State* [A5.5.3]

a. Live load moment due to fatigue truck (*FTr*) at midspan (Fig. E7.4-10)

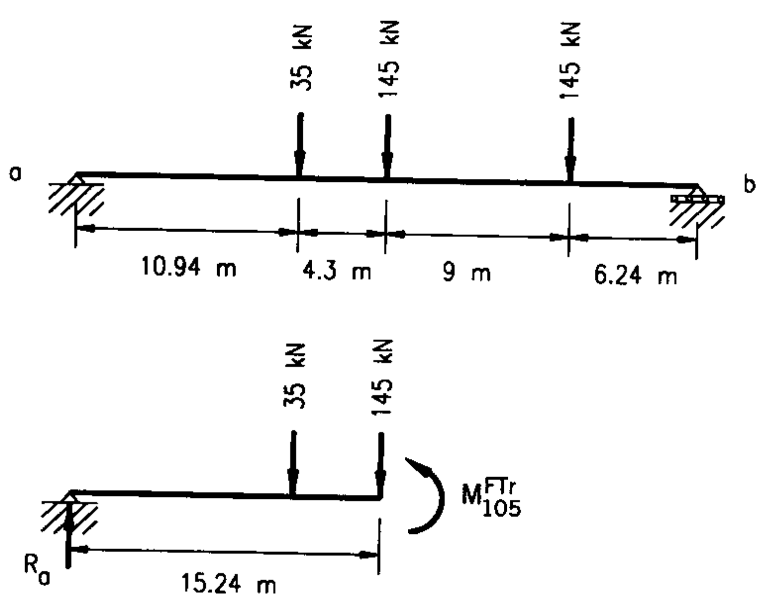


Fig. E7.4-10 Fatigue truck placement for maximum positive moment at midspan.

$$R_a = 145\left(\frac{6.240 + 15.240}{30.480}\right) + 35\left(\frac{19.540}{30.480}\right) = 124.6 \text{ kN}$$

$$M_{105}^{FTr} = [(124.6)(15.240) - (35)(4.300)] = 1748.4 \text{ kN m}$$

Exterior Girder DF—remove 1.2 multiple presence for fatigue

$$g_M^{SE} = \frac{0.762}{1.2} = 0.635$$

Factored Moment including $IM = 15\%$

$$M_{\text{fatigue}} = 0.75(0.635)(1748.4)(1.15) = 957.6 \text{ kN m}$$

b. *Dead load moments at midspan*

Exterior girder	(Table E7.4-4)
Noncomp.	$M_{DC1} = 2900$ kN m
Comp.	$M_{DC2} + M_{DW} = (540 + 353) = 893$ kN m

If section is in compression under DL and two times fatigue load, fatigue does not need to be investigated [A5.5.3.1].

$$f_b = -\frac{F_f}{A_g} - \frac{F_t e_{CL}}{S_{bg}} + \frac{M_{DC1}}{S_{bg}} + \frac{M_{DC2} + M_{DW} + 2M_{\text{fatigue}}}{S_{bc}}$$

$$= -\frac{5.14 \times 10^6}{486\,051} - \frac{(5.14 \times 10^6)(489)}{207.3 \times 10^6} + \frac{2900 \times 10^6}{207.3 \times 10^6} + \frac{[893 + 2(957.6)]10^6}{290.4 \times 10^6}$$

= 0.94 MPa, tension; therefore,

fatigue shall be considered

Stress range due only to $M_{\text{fatigue}} = 957.6$ kN m

Section Properties: Cracked Section properties [A5.5.3.1] if

$1.5M_{\text{fatigue}}$ and tensile stress exceeds $0.25\sqrt{f'_c}$

$$= 0.25\sqrt{55} = 1.85 \text{ MPa}$$

$$f_b = -\frac{5.14 \times 10^6}{486\ 051} - \frac{(5.14 \times 10^6)(489)}{207.3 \times 10^6} + \frac{2900 \times 10^6}{207.3 \times 10^6} + \frac{[893 + 1.5(957.6)]10^6}{290.4 \times 10^6}$$

$$= -0.71 \text{ MPa} < 1.85 \text{ MPa; therefore,}$$

use gross section properties

Concrete stress at *cg* of prestress tendons due to fatigue load

$$f_{cgp} = \frac{M_{\text{fatigue}}(977 - 119)}{283.7 \times 10^9} = \frac{957.6 \times 10^6}{330.7 \times 10^6} = 2.9 \text{ MPa}$$

Stress in tendon due to fatigue load (*FL*)

$$f_{pFL} = f_{cgp}\frac{E_p}{E_c} = (2.9)\left(\frac{197\ 000}{35\ 600}\right) = 16.0 \text{ MPa}$$

Stress range in prestressing tendons shall not exceed (Table 7.9) [A5.5.3.3]

- 125 MPa for radii of curvature greater than 9000 mm
- 70 MPa for radii of curvature less than 3600 mm

Harped Tendons (Fig. E7.4-11)

$$e_{\text{end}} = 258 \text{ mm}, \Delta y = 489 - 258 = 231 \text{ mm},$$

$$e_{0.33L} = e_{CL} = 489 \text{ mm}$$

At hold down point, the radius of curvature depends on the hold down device and is small, therefore $R < 3600$ mm.

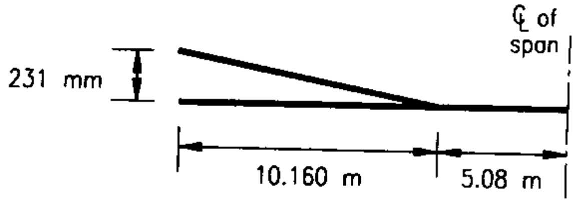


Fig. E7.4-11 Schematic of harped tendons.

$$f_{pFL} = 16.0 \text{ MPa} < 70 \text{ MPa}, \qquad \text{OK}$$

Satisfies fatigue limit state.

8. *Calculate Deflection and Camber*

a. Immediate deflection due to live load and impact (Fig. E7.4-12)

$$\Delta(x < a)x = \frac{Pbx}{6EIL}(L^2 - b^2x^2),\ b = L - a$$

$$\Delta x\left(x = \frac{L}{2}\right) = \frac{PL^3}{48EI}$$

Use EI for $f'_c = 55$ MPa and composite section

$$E_c = 35\,600 \text{ MPa},\ I_c = 283.7 \times 10^9 \text{ mm}^4$$

$$E_cI_c = 10.1 \times 10^{15} \text{ N mm}^2 = 10.1 \times 10^6 \text{ kN m}^2$$

<u>P = 35 kN</u>, $x = 15.240$ m, $a = 19.540$ m, $b = 10.940$ m

$$\Delta_x^{35} = \frac{(35)(10.940)(15.240)}{6(EI)(30.480)}(30.480^2 - 10.940^2 - 15.240^2)$$

$$= \frac{18.4 \times 10^3}{EI} = \frac{18.4 \times 10^3}{10.1 \times 10^6} = 0.002 \text{ m} = 2 \text{ mm}$$

<u>P = 145 kN</u>, $x = a = b = 15.240$ m

$$\Delta_x^{145} = \frac{(145)(30.480)^3}{48EI} = \frac{85.5 \times 10^3}{EI}$$

$$= \frac{85.5 \times 10^3}{10.1 \times 10^6} = 0.008 \text{ m} = 8 \text{ mm}$$

<u>P = 145 kN</u>, $x = 15.240$ m, $a = 19.540$ m, $b = 10.940$ m

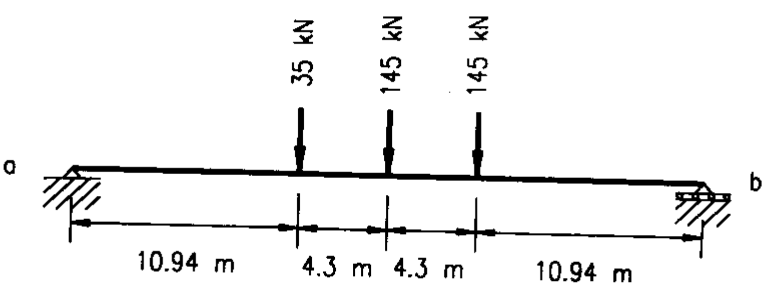


Fig. E7.4-12 Live load placement for deflection at midspan.

$$\Delta_x^{145'} = \frac{(145)(10.940)(15.240)}{6EI(30.480)}(30.480^2 - 10.940^2 - 15.240^2)$$

$$= \frac{76.3 \times 10^3}{EI} = \frac{76.3 \times 10^3}{10.1 \times 10^6} = 0.008 \text{ m} = 8 \text{ mm}$$

Total deflection due to truck

$$\Delta_{105}^{Tr} = \frac{(18.4 + 85.5 + 76.3) \times 10^3}{EI} = \frac{180.2 \times 10^3}{10.1 \times 10^6}$$

$$= 0.018 \text{ m} = 18 \text{ mm}$$

Deflection DF $= N_L/N_G = 3/6 = 0.5$, $IM = 33\%$

$$\Delta_{105}^{L+I} = 0.5(18)(1.33) = 12 \text{ mm} \downarrow$$

$$= 12 \text{ mm} \leq \frac{L}{800} = \frac{30\,480}{800} = 38 \text{ mm}, \qquad \text{OK}$$

b. Long-term deflections (Collins and Mitchell, 1991) Loads on exterior girder from Section 7.10.4, Part H.2.

- Elastic deflections due to girder self weight at release of prestress

$$E_{ci} = 30\,360 \text{ MPa}, \qquad I_g = 126\,011 \times 10^6 \text{ mm}^4$$

$$E_{ci}I_g = 3.83 \times 10^6 \text{ kN m}^2$$

$$\Delta_{gi} = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\frac{(11.44)(30.480)^4}{3.83 \times 10^6}$$

$$= 0.034 \text{ m} = 34 \text{ mm} \downarrow$$

- Elastic camber due to prestress at time of release for double harping point with $\beta L = 0.333L$ (Collins and Mitchell, 1991)

$$\Delta_{pi} = \left[\frac{e_c}{8} - \frac{\beta^2}{6}(e_c - e_e)\right]\frac{F_iL^2}{EI}$$

$$= \left[\frac{489}{8} - \frac{(0.333)^2}{6}(489 - 258)\right]\frac{(6346)(30.480)^2}{3.83 \times 10^6}$$

$$= 88 \text{ mm} \uparrow$$

At release, net upward deflection:

$$88 - 34 = 54 \text{ mm} \uparrow$$

- Elastic deflection due to deck and diaphragms on exterior girder

$$DC1 - w_g = 24.13 - 11.44 = 12.69 \text{ N/mm} = 12.69 \text{ kN/m}$$

Diaphragm = 9.705 kN

$$E_c = 35\,600 \text{ MPa}$$

$$E_c I_g = 4.49 \times 10^6 \text{ kN} \cdot \text{m}^2$$

$$b = \frac{L}{3} = 10.160 \text{ m}$$

$$\Delta_{DC} = \frac{5}{384} \frac{wL^4}{EI} + \frac{Pb}{24EI} (3L^2 - 4b^2)$$

$$= \frac{5}{384} \frac{(12.69)(30.480)^4}{4.49 \times 10^6}$$

$$+ \frac{(9.705)(10.160)}{24(4.49 \times 10^6)} [3(30.480)^2 - 4(10.160)^2]$$

$$= 0.032 + 0.002 = 0.034 \text{ m} = 34 \text{ mm} \downarrow$$

- Elastic deflection due to additional dead load acting on composite section

$$DW + \text{Barrier} = 3.04 + 4.65 = 7.69 \text{ N/mm} = 7.69 \text{ kN/m}$$

$$\Delta_c = \frac{5}{384} \frac{wL^4}{EI} = \frac{5}{384} \frac{(7.69)(30.480)^4}{10.1 \times 10^6} = 0.009 \text{ m} = 9 \text{ mm} \downarrow$$

Note the full barrier load is conservatively applied to the exterior girder. Many designers distribute this load equally to all girders.

Long-Term Deflections

Using the multipliers in Table E7.4-5 (PCI, 1992) to approximate the creep effect, the net upward deflection at the time the deck is placed is

$$\Delta_1 = 1.80(88) - 1.85(34) = 96 \text{ mm} \uparrow$$

TABLE E7.4-5 Suggested Multipliers To Be Used as a Guide in Estimating Long-Time Cambers and Deflections for Typical Members[a]

	Without Composite Topping	With Composite Topping
At erection		
1. Deflection (downward) component—apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
2. Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
Final		
3. Deflection (downward) component—apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40
4. Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
5. Deflection (downward)—apply to elastic deflection due to superimposed dead load only	3.00	3.00
6. Deflection (downward)—apply to elastic deflection caused by the composite topping		2.30

[a]In PCI Table 4.6.2. [From *PCI Design Handbook: Precast and Prestressed Concrete,* 4th ed., Copyright © 1992 by the Precast/Prestressed Concrete Institute, Chicago, IL.]

The net long-term upward deflection is

$$\Delta_{LT} = 2.20(88) - 2.40(34) - 2.30(34) - 3.00(9) = 7 \text{ mm} \uparrow$$

After construction has been completed the center of the bridge is estimated to creep downward from an initial upward deflection of 96 mm to a final upward value of 7 mm.

J. *Investigate Strength Limit State*

1. *Flexure*

a. Stress in prestressing steel bonded tendons (Eq. 7.59) [A5.7.3.1.1]

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

where (Eq. 7.60)

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2\left(1.04 - \frac{1674}{1860}\right) = 0.28$$

By using transformed section (Fig. E7.4-8)

$$b = 1635.4 \text{ mm}, d_p = (1350 + 50 + 190) - 119 = 1471 \text{ mm}$$

$$f'_c = 55 \text{ MPa}, A_s = A'_s = 0$$

$$\beta_1 = 0.85 - \frac{55 - 28}{7}(0.05) = 0.657$$

from Eq. 7.68

$$c = \frac{A_{ps}f_{pu} + A_s f_y + A'_s f'_y - 0.85\beta_1 f'_c(b - b_w)h_f}{0.85 f'_c \beta_1 b_w + kA_{ps}(f_{pu}/d_p)}$$

$$= \frac{(5133)(1860) - 0.85(0.657)(55)(1635.4 - 150)(190)}{0.85(55)(0.657)(150) + 0.28(5133)(1860/1471)}$$

$$= 136.8 \text{ mm}$$

$$f_{ps} = 1860\left[1 - 0.28\left(\frac{136.8}{1471}\right)\right] = 1812 \text{ MPa}$$

$$T_p = A_{ps}f_{ps} = \frac{(5133)(1812)}{10^3} = 9300 \text{ kN}$$

b. Factored Flexural Resistance—Flanged Sections [A5.7.3.2.2]

$$a = \beta_1 c = (0.657)(136.8 \text{ mm}) = 89.9 \text{ mm}$$

$$\phi = 1.0$$

from Eq. 7.76

$$\phi M_n = \phi\left[A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A'_s f'_y\left(d'_s - \frac{a}{2}\right) + 0.85\beta_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)\right]$$

$$= 1.0\left[5133(1812)\left(1471 - \frac{89.9}{2}\right) + 0.85(0.657)(55)(1635.4 - 150)(190)\left(\frac{89.9}{2} - \frac{190}{2}\right)\right]$$

$$\phi M_n = 12.8 \times 10^9 \text{ N mm} = 12\,800 \text{ kN m}$$

$$M_u = 9477 \text{ kN m (Table E7.4-4)}$$

$$\phi M_n > M_u, \quad \text{OK}$$

c. Limits for Reinforcement [A5.7.3.3]

- Maximum reinforcement limited by (Eq. 7.89)

$$\frac{c}{d_e} \le 0.42 \quad \text{for} \quad d_e = d_p$$

$$\frac{c}{d_p} = \frac{136.8}{1471} = 0.093 < 0.42, \quad \text{OK}$$

- Minimum reinforcement limited by (Eq. 7.90)

$$\phi M_n \ge 1.2 M_{cr}$$

$$M_{cr} \quad \text{based on} \quad f_r = 0.63\sqrt{f'_c} = 0.63\sqrt{55} = 4.67 \text{ MPa}$$

Under final conditions at midspan with Service *III* live load M_L = 2353 kN m, the bottom tensile stress is f_{bf} = 2.44 MPa. To cause cracking, an additional tensile stress of $\Delta f_b = 4.67 - 2.44 = 2.23$ MPa is required. Additional moment to cause this stress is

$$\Delta M = S_{bc}\Delta f_b = (290.4 \times 10^6)(2.23)$$

$$= 648 \times 10^6 \text{ N mm} = 648 \text{ kN m}$$

Addition of ΔM with the service dead and live load moments yields

$$M_{cr} = (2900 + 893 + 2353 + 648) = 6794 \text{ kN m}$$

$$1.2M_{cr} = 1.2(6794) = 8153 \text{ kN m}$$

$$\phi M_n > 1.2M_{cr}, \quad \text{OK}$$

52 − 12.70 mm low-relaxation strands satisfy strength limit state

2. *Shear* [A5.8]

a. General

$$\phi_v = 0.9 \qquad [\text{A5.5.4.2.1}]$$

$$\eta = 0.95$$

$$V_n = V_c + V_s + V_p \le 0.25 f'_c b_v d_v \text{ [A5.8.3.3]}$$

$$d_e = 1471 \text{ mm}$$

$$d_v = d_e - \frac{a}{2}$$

$$\geq \max \begin{cases} 0.9d_e = 0.9(1471) = 1324 \text{ mm, governs at } CL \\ 0.72h = 0.72(1590) = 1145 \text{ mm} \end{cases}$$

$$\text{At } CL\text{: } a = \beta_1 c = (0.657)(136.8) = 89.9 \text{ mm}$$

$$d_v = 1471 - \frac{89.9}{2} = 1426 \text{ mm [A5.8.2.7]}$$

At the end of the beam

$$d_e = 1590 - 350 = 1240 \text{ mm}$$

$$d_v = \max \begin{cases} 0.9d_e = 0.9(1240) = 1116 \text{ mm} \\ 0.72h = 0.72(1590) = 1145 \text{ mm, governs} \end{cases}$$

$$b_v = \text{minimum web width within } d_v = 150 \text{ mm}$$

$$f'_c\text{(girder)} = 55 \text{ MPa}$$

b. Prestress contribution to shear resistance

V_p = vertical component of prestressing force

Transfer length = 60 strand diameters = 60(12.70) = 762 mm [A5.8.2.3]

Critical section for shear $\geq 0.5 d_v \cot\theta$ or d_v = 1145 mm [A5.8.3.2]

$d_v >$ transfer length, therefore, full value of V_p can be used.

cg of harped strands (12) = 1350 − 175 = 1175 mm from bottom of girder

= 1175 − 119 = 1056 mm from cg A_{ps} at

centerline (Fig. E7.4-13)

$$\gamma = \tan^{-1} \frac{1056}{10\ 160} = 5.934°$$

$$F_f = 5140 \text{ kN}$$

$$V_p = \frac{12}{52} F_f \sin\gamma = \frac{12}{52}(5140) \sin 5.934 = 122.6 \text{ kN}$$

c. Design for shear Design for shear at a distance of d_v from the support and at tenth points along the span. Calculations are

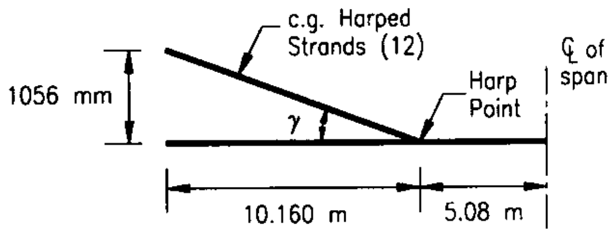


Fig. E7.4-13 Harped tendon profile.

shown below for a distance d_v from the support (Fig. E7.4-14) and Location 101. The same procedure is used for the remaining points with final results given later in Table E7.4-6 of Section 7.10.4, Part J.2.e.

$$d_v = 1145 \text{ mm}$$

$$\xi = \frac{d_v}{L} = \frac{1145}{30\,480} = 0.0376$$

For a unit load, $w = 1.0$ N/mm $= 1.0$ kN/m

$$V_x = wL(0.5 - \xi) = w30.480(0.5 - 0.0376) = 14.094w \text{ kN}$$

$$M_x = 0.5wL^2(\xi - \xi^2) = 0.5w(30.480)^2(0.0376 - 0.0376^2)$$

$$= 16.8w \text{ kN m}$$

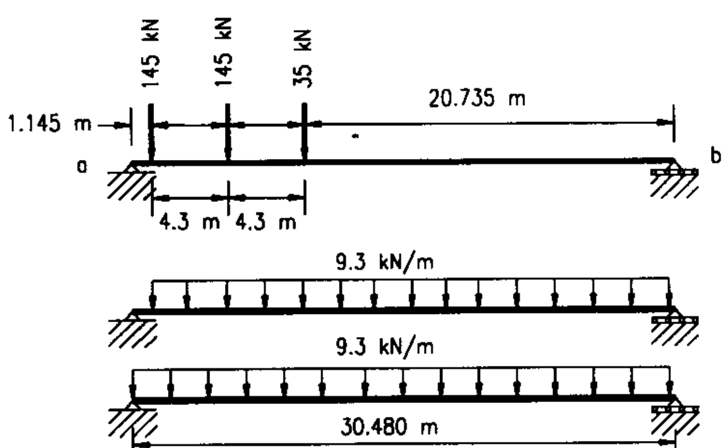


Fig. E7.4-14 Live load placement for maximum shear and moment at Location 100.376.

Exterior girders dead loads taken from Section 7.10.4, Part H.2

$$DC_1 = 24.13 \text{ N/mm}$$

$$DC_2 = 4.65 \text{ N/mm}$$

$$DW = 3.04 \text{ N/mm}$$

$$DIAPH = 9.705 \text{ kN}$$

$$IM = 0.33$$

$$V_{100.376}^{Tr} = \left[145\left(\frac{25.035 + 29.335}{30.480}\right) + 35\left(\frac{20.735}{30.480}\right)\right]$$

$$= 282.5 \text{ kN}$$

$$M_{100.376}^{Tr} = 1.145(282.5) = 323.5 \text{ kN m}$$

$$V_{100.376}^{Ln} = \frac{1}{2}(9.3)\left(\frac{29.335}{30.480}\right)^2 = 131.3 \text{ kN}$$

$$M_{100.376}^{Ln} = \tfrac{1}{2}(1.145)(29.335)(9.3) = 156.2 \text{ kN m}$$

$$V_u = \eta[1.25DC + 1.50DW + 1.75(LL + IM)]$$

$$= 0.95\{1.25[(24.13 + 4.65)(14.094) + 9.705]$$
$$+ 1.50[(3.04)(14.094)] + 1.75(0.762)[(282.5)(1.33)$$
$$+ 131.3]\}$$
$$= 1197 \text{ kN}$$

$$M_u = 0.95\{1.25[(24.13 + 4.65)(16.8) + 9.705(1.145)]$$
$$+ 1.50[(3.04)(16.8)] + 1.75(0.762)[(323.5)(1.33)$$
$$+ (156.2)]\}$$
$$= 1403 \text{ kN m}$$

From Eq. 7.159 [A5.8.3.4.2]

$$v = \frac{V_u - \phi V_p}{\phi b_v d_v}$$

$$= \left[\frac{(1.197 \times 10^6) - 0.9(122.6 \times 10^3)}{0.9(150)(1145)}\right]$$

$$= 7.02 \text{ MPa}$$

$$\frac{v}{f'_c} = \frac{7.02}{55} = 0.128 > 0.100$$

$$\text{therefore, } s_{max} = \min\begin{cases} 0.4d_v = 0.4(1145) = 458 \text{ mm} \\ 300 \text{ mm, governs} \end{cases} \quad [\text{A5.8.2.7}]$$

$$d_e = d_p = 1590 - 350 + \frac{1145}{10\,160}(489 - 258) = 1266 \text{ mm}$$

1st Iteration

Assume $\theta = 30°$, $f_{po} \approx f_{se} = 1001$ MPa

$$d_e = 1266 \text{ mm}$$

$$d_v = \max\begin{cases} d_e - \dfrac{a}{2} = 1266 - \dfrac{89.9}{2} = 1221 \text{ mm, governs} \\ 0.9d_e = 0.9(1266) = 1139 \text{ mm} \\ 0.72h = 0.72(1590) = 1145 \text{ mm} \end{cases}$$

$$d_v = 1221 \text{ mm}$$

From Eq. 7.170 [A5.8.3.4.2]

$$\varepsilon_x = \frac{(M_u/d_v) + 0.5N_u + 0.5V_u \cot\theta - A_{ps}f_{po}}{E_sA_s + E_pA_{ps}}$$

$$= \frac{(1.402 \times 10^9/1221) + 0.5(1.197 \times 10^6)\cot 30 - (5133)(1001)}{(197\,000)(5133)}$$

$$= -2.9 \times 10^{-3} \text{ (compression)}$$

Because ε_x is negative, it shall be reduced by the factor [A5.8.3.4.2]

$$F_\varepsilon = \frac{E_sA_s + E_pA_{ps}}{E_cA_c + E_sA_s + E_pA_{ps}} = \frac{E_pA_{ps}}{E_cA_c + E_pA_{ps}}$$

Where A_c is the area of concrete on flexural tension side of member defined as concrete below $h/2$ of member [Fig. A5.8.3.4.2-3]

$$h = 1350 + 50 + 190 = 1590 \text{ mm}, \quad \frac{h}{2} = 795 \text{ mm}$$

$$A_c \cong (135)(975) + 2(\tfrac{1}{2})(140)(412.5) + (660)(150)$$

$$\approx 288\,375 \text{ mm}^2$$

$$E_c = 35\ 600 \text{ MPa}$$

$$F_\varepsilon = \frac{(197\ 000)(5133)}{(35\ 600)(288\ 375) + (197\ 000)(5133)} = 0.0897$$

$$\varepsilon_x = (-0.0029)(0.0897) = -0.26 \times 10^{-3}$$

Using $v/f'_c = 0.128$ and ε_x with Figure 7.43 $\Rightarrow \theta = 20°$

$$\cot\theta = 2.747 \qquad \beta = 2.75$$

2nd Iteration

$$\theta = 20°$$

$$M_{DC1} = (24.13)(16.8 \times 10^6) + (9705)(1221)$$

$$= 416.4 \times 10^6 \text{ N mm}$$

$$e = d_p - (y_{tg} + t_s + 50)$$

$$= 1221 - (742 + 190 + 50) = 239 \text{ mm}$$

$$f_{pc} = -\frac{F_f}{A_g} + \frac{F_f e(y_{bc} - y_{bg})}{I_g} - \frac{M_{DC1}(y_{bc} - y_{bg})}{I_g}$$

$$= -\frac{5.14 \times 10^6}{486\ 051} + \frac{(5.14 \times 10^6)(239)(977 - 608)}{126\ 011 \times 10^6}$$

$$- \frac{(416.4 \times 10^6)(977 - 608)}{126\ 011 \times 10^6}$$

$$f_{pc} = -8.20 \text{ MPa}$$

$$f_{po} = f_{se} + f_{pc}\frac{E_p}{E_c} = 1001 + 8.2\,\frac{197\ 000}{35\ 600}$$

$$= 1046.5 \text{ MPa (precompression)}$$

$$\varepsilon_x = \frac{\dfrac{1.402 \times 10^9}{1221} + 0.5(1.196 \times 10^6)(2.747) - (5133)(1046.5)}{(197\ 000)(5133)}$$

$$= -0.0026$$

$$F_\varepsilon \varepsilon_x = 0.0897(-0.0026) = -0.23 \times 10^{-3}$$

Figure 7.43 $\Rightarrow \theta = 20°$, converged,

use $\cot\theta = 2.747 \qquad \beta = 2.75$

$$V_c = 0.083\beta\sqrt{f'_c}\,b_v d_v$$

$$= 0.083(2.75)\sqrt{55}(150)(1221) = 310 \times 10^3 \text{ N}$$

Required

$$V_s = \frac{V_u}{\phi} - V_c = \frac{1.196 \times 10^6}{0.9} - 310 \times 10^3 = 1.019 \times 10^6 \text{ N}$$

Spacing of No. 15 stirrups, (Eq. 7.172)

$$d_s = 16 \text{ mm}, \qquad A_v = 2(200 \text{ mm}^2) = 400 \text{ mm}^2$$

$$s \le \frac{A_v f_y d_v \cot\theta}{V_s} = \frac{(400)(400)(1221)(2.747)}{1.019 \times 10^6}$$

$$s \le 526 \text{ mm} > s_{\max} = 300 \text{ mm}$$

Check Longitudinal Reinforcement (Eq. 7.169) [A5.8.3.5]

$$A_s f_y + A_{ps} f_{ps} \ge \left[\frac{M_y}{d_v \phi_f} + 0.5\frac{N_u}{\phi_a} + \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p\right)\cot\theta\right]$$

$$V_s = 1.019 \times 10^6 \left[\frac{526}{300}\right] = 1.787 \times 10^6 \text{ N}$$

$$(5133)(1812) \ge \frac{1.402 \times 10^9}{(1221)(1.0)} + \left[\frac{1.196 \times 10^6}{0.9} - 0.5(1.787 \times 10^6) - 122.6 \times 10^3\right] 2.747$$

$$9.301 \times 10^6 \text{ N} > 2.007 \times 10^6 \text{ N}, \qquad \text{OK}$$

Use s = 300 mm at Location 100.38

d. Location 101

$$d_e = 1590 - 350 + \frac{3048}{10\,160}(489 - 258) = 1309 \text{ mm}$$

$$d_v = \max\begin{cases} d_e - \dfrac{a}{2} = 1309 - \dfrac{89.9}{2} = 1264 \text{ mm, governs} \\ 0.9d_e = 0.9(1309) = 1178 \text{ mm} \\ 0.72h = 0.72(1590) = 1145 \text{ mm} \end{cases}$$

$$d_v = 1264 \text{ mm}$$

$$V_u = 1067.9 \text{ kN}, \qquad M_u = 3481.4 \text{ kN m}$$

$$v = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{[1067.9 - 0.9(122.6)]10^3}{0.9(150)(1264)} = 5.61 \text{ MPa}$$

$$\frac{v}{f'_c} = \frac{5.61}{55} = 0.102 > 0.100, \text{ therefore, } s_{\max} = 300 \text{ mm}$$

1st Iteration

$$\text{Assume } \theta = 28°, \qquad f_{po} = 1050 \text{ N/mm}^2$$

$$\varepsilon_x = \frac{\dfrac{3481.4 \times 10^6}{1264} + 0.5(1067.9 \times 10^3) \cot 28 - (5133)(1050)}{(197\,000)(5133)}$$

$$= -0.0016 \qquad \text{(compression)}$$

$$F_\varepsilon \varepsilon_x = (0.0897)(-0.0016) = -0.145 \times 10^{-3}$$

$$\text{Figure 7.43} \Rightarrow \theta = 23.5°, \qquad \cot\theta = 2.300$$

2nd Iteration

$$M_{DC1} = 1036.1 \text{ kN m}$$

$$e = 1309 - (742 + 50 + 190) = 327 \text{ mm}$$

$$f_{pc} = \frac{5.14 \times 10^6}{486\,051} + \frac{(5.14 \times 10^6)(327)(369)}{126\,011 \times 10^6}$$

$$- \frac{(1036.1 \times 10^6)(369)}{126\,011 \times 10^6} = -8.69 \text{ MPa}$$

$$f_{po} = 1001 + 8.69\left(\frac{197\,000}{35\,600}\right) = 1049 \text{ MPa (precompression)}$$

$$\varepsilon_x = \frac{\dfrac{3481.4 \times 10^6}{1264} + 0.5(1067.9 \times 10^3)(2.300) - (5133)(1049)}{(197\,000)(5133)}$$

$$= -1.387 \times 10^{-3}$$

$$F_\varepsilon \varepsilon_x = (0.0897)(-1.387 \times 10^{-3}) = -0.124 \times 10^{-3}$$

$$\text{Figure 7.43} \Rightarrow \theta = 23.5°, \qquad \beta = 5, \qquad \cot\theta = 2.300$$

$$V_c = 0.083\beta\sqrt{f'_c}\, b_v d_v$$

$$= 0.083(5)\sqrt{55}(150)(1264) = 583.5 \times 10^3 \text{ N}$$

Requires

$$V_s = \frac{V_u}{\phi} - V_c = \frac{1067.9 \times 10^3}{0.9} - 583.5 \times 10^3$$

$$= 603.1 \times 10^3 \text{ N}$$

$$s \leq \frac{(400)(400)(1264)(2.300)}{603.1 \times 10^3} = 771 \text{ mm} > s_{max}$$

$$s_{max} = 300 \text{ mm}$$

$$V_s = 603.1 \times 10^3 \left(\frac{771}{300}\right) = 1550 \times 10^3 \text{ N}$$

Check Longitudinal Reinforcement

$$(5133)(1812) \geq \frac{3481.4 \times 10^6}{(1264)(1.0)} + \left(\frac{1067.9 \times 10^3}{0.9} - 0.5(1550 \times 10^3) - 122.6 \times 10^3\right) 2.300$$

$$9.301 \times 10^6 \text{ N} \geq 3.419 \times 10^6 \text{ N}, \qquad \text{OK}$$

Use s = 300 mm at Location 101

e. Summary of shear design (Table E7.4-6)

f. Horizontal shear [A5.8.4] At interface between two concretes cast at different times the nominal shear resistance shall be taken as

$$V_{nh} = cA_{cv} + \mu(A_{vf}f_y + P_c) \begin{cases} \leq 0.2f'_cA_{cv} \\ \leq 5.5A_{cv} \end{cases}$$

where

A_{cv} = area of concrete engaged in shear transfer

$= (1225 \text{ mm})(1 \text{ mm}) = 1225 \text{ mm}^2$

A_{vf} = area of shear reinforcement crossing the shear plane

$= 2(200 \text{ mm}^2) = 400 \text{ mm}^2, \qquad 2 \text{ legs}$

f_y = yield strength of reinforcement = 400 MPa

TABLE E7.4-6 Summary of Shear Design

	Location					
	100.38	101	102	103	104	105
V_u (kN)	1196	1068	865	666	459	268
M_u (kN m)	1.4	3481	6162	8036	9159	9470
V_p (kN)	122.6	122.6	122.6	122.6	0	0
d_v (mm)	1221	1204	1334	1403	1426	1426
$\frac{v}{f'_c}$	0.128	0.102	0.0762	0.0533	0.043	0.0253
f_{po} (MPa)	1047	1049	1056	1060	1064	1066
$\varepsilon_x \times 10^3$	−0.23	−0.124	0.0469	0.781	1.236	1.320
θ, degrees	20	23.5	27	34	38.5	39
β	2.75	5	4.4	2.3	2.1	2.1
V_c (kN)	310	584	296	246	230	230
Required V_s (kN)	1019	603	420	442	224	209
s (mm)	526	771	999	752	1281	13 482
$A_{ps}f_{ps} = 9.301 \times 10^6$ N $\geq$ ($\times 10^6$ N)	2.007	3.419	5.580	6.233	6.764	6.719
Prov'd s (mm)	300	300	600	600	600	600
$\phi_f = 1.0$, $\phi_v = 0.9$						
$\frac{M_u}{(\phi_f d_v)}$ ($\times 10^6$)	1.148	2.754	4.619	5.728	6.423	6.641
$\frac{V_u \cot\theta}{\phi_v}$ ($\times 10^6$)	3.650	2.729	1.887	1.098	0.642	0.367
$-0.5V_s \cot\theta$ ($\times 10^6$)	−2.454	−1.783	−0.685	−0.411	−0.300	−0.290
$-V_p \cot\theta$ ($\times 10^6$)	−0.337	−0.282	−0.241	−0.182	0	0
Σ last 3 rows ($\times 10^6$)	0.859	0.665	0.961	0.505	0.342	0.077
Σ last 4 rows ($\times 10^6$)	2.007	3.419	5.580	6.233	6.764	6.719

$$\left.\begin{array}{l} c = \text{cohesion factor} = 0.70 \text{ MPa} \\ \mu = \text{friction factor} = 1.0 \end{array}\right\}$$

intentionally roughened [A5.8.4.2]

P_c = permanent net compressive force normal to shear plane

= overhang + slab + haunch + barrier

= [5.37 + 5.83 + 0.61 + 4.65] = 16.46 N/mm

$$V_{nh} = 0.70(1225) + 1.0\left[\left(\frac{400}{s}\right)(400) + 16.46\right] \quad \text{(E7.4-1)}$$

$$= 873.96 + \frac{160\,000}{s} \text{ N/mm}$$

s = spacing of shear reinforcement, mm

$$V_{nh} \leq \min\begin{cases} 0.2f'_cA_{cv} = 0.2(30)(1225) = 7350 \text{ N/mm} \\ 5.5A_{cv} = 5.5(1225) = 6740 \text{ N/mm, governs} \end{cases}$$

$$\phi_v V_{nh} \geq \eta V_{uh}$$

where

V_{uh} = horizontal shear due to barrier, *FWS* and *LL* + *IM*

$$= \frac{V_u Q}{I_c}$$

$$I_c = 283.7 \times 10^9 \text{ mm}^4$$

$$Q = A\bar{y} = (1635.4)(190)\left(1590 - 977 - \frac{190}{2}\right)$$

$$= 160.96 \times 10^6 \text{ mm}^3$$

$$V_u = 1.25DC2 + 1.50DW + 1.75(LL + IM)$$

Location 100 (Table E7.4-4)

$$V_u = 1.25(71) + 1.50(46) + 1.75(407) = 870 \text{ kN}$$

$$V_{uh} = \frac{(870 \times 10^3)(160.96 \times 10^6)}{283.7 \times 10^9} = 493.4 \text{ N/mm}$$

$$\frac{\eta V_{uh}}{\phi_v} = \frac{0.95(493.4)}{0.9} = 520.8 \text{ N/mm} < 6740 \text{ N/mm} \quad \text{(E7.4-2)}$$

Equating Eqs. E7.4-1 and E7.4-2

$$873.96 + \frac{160\ 000}{s} \geq 520.8$$

$$s \leq \frac{160\ 000}{873.96 - 520.8} = 453 \text{ mm}$$

$$s_{\max} = 300 \text{ mm}$$

Use s = 300 mm at Location 100

At Location 100.38 Interpolating between Locations 100 and 101

$$V_u = 1.25(65.7) + 1.50(42.6) + 1.75(386.5) = 822.4 \text{ kN}$$

$$V_{uh} = \frac{(822.4 \times 10^3)(160.96 \times 10^6)}{283.7 \times 10^9} = 466.5 \text{ N/mm}$$

$$\frac{\eta V_{uh}}{\phi_v} = \frac{0.95(466.5)}{0.9} = 492.4 \text{ N/mm} < 6740 \text{ N/mm}$$

$$s \leq \frac{160\ 000}{873.96 - 486.1} = 413 \text{ mm} > s_{\max} = 300 \text{ mm}$$

Use s = 300 mm at Location 100.38

At Location 101 (Table E7.4-4)

$$V_u = 1.25(57) + 1.50(37) + 1.75(353) = 745 \text{ kN}$$

$$V_{uh} = \frac{(745 \times 10^3)(160.96 \times 10^6)}{283.7 \times 10^9} = 422.7 \text{ N/mm}$$

$$\frac{\eta V_{uh}}{\phi_v} = \frac{0.95(422.7)}{0.9} = 446.2 \text{ N/mm}$$

$$s \leq \frac{160\ 000}{873.96 - 446.2} = 374 \text{ mm} > s_{\max}$$

$$s_{\max} = 300 \text{ mm}$$

Use s = 300 mm at Location 101

By inspection, horizontal shear does not govern strand spacing for any of these or remaining locations.

g. Check details

Anchorage Zone [A5.10.10]

The factored resistance provided by transverse reinforcement P_r shall not be less than 4% of the factored prestressing force [A3.4.3] [1.3 times jacking force = $1.3(6.34 \times 10^6) = 8.24 \times 10^6$ N]

$$P_r = \phi_{ca} f_y A_s$$

where

ϕ_{ca} = 0.80 (compression in anchorage zones) (Table 7.10) [A5.5.4.2.1]

f_y = 400 MPa

A_s = total area of transverse reinforcement within $d/4$ of end of beam

d = depth of precast beam = 1350 mm

$$\phi_{ca} f_y A_s \geq 0.04 F_{ui} = 0.04(8.24 \times 10^6) = 329.6 \times 10^3 \text{ N}$$

$$A_s \geq \frac{329.6 \times 10^3}{0.8(400)} = 1030 \text{ mm}^2$$

$$\text{within } \frac{d}{4} = \frac{1350}{4} = 337.5 \text{ mm}$$

Number of No. 15 U stirrups required: $\frac{1030}{400} = 2.6$

Use 3 No. 15 stirrups at 130 mm

Confinement Reinforcement: [A5.10.10.2]

For a distance of $1.5d = 1.5(1350) = 2025$ mm from the end of the beam, reinforcement shall be placed to confine the prestressing steel in the bottom flange.

Use 14 No. 10 at 150 mm shaped to enclose the strands

K. ***Design Sketch*** The design of the prestressed concrete girder is summarized in Figure E7.4-15. The design utilized the NU 1350 girder shape developed by Nebraska University, f'_c = 55 MPa, and f'_{ci} = 40 MPa. The prestressing steel consists of 52 − 1860 MPa, low-relaxation 12.70 mm, seven-wire strands.

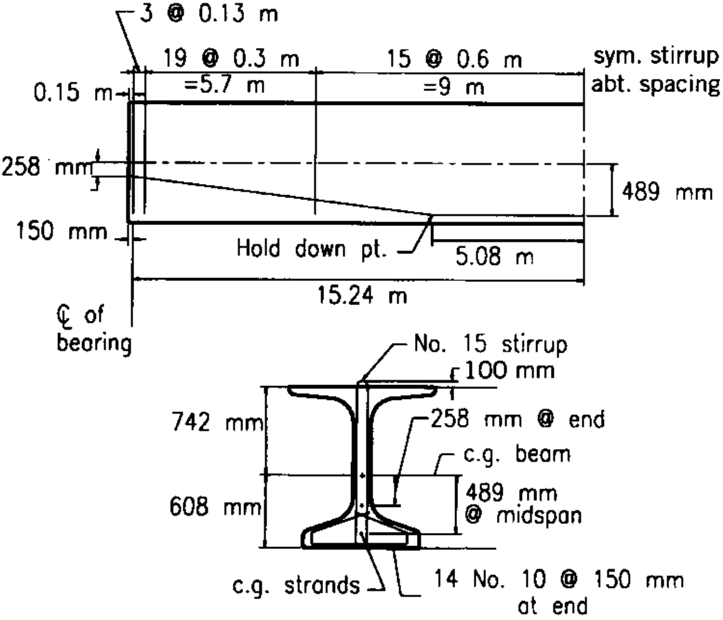


Fig. E7.4-15 Design sketch for prestressed girder.