3.4 LRFD PPC I-Beam and Bulb T-Beam Design

This design guide focuses on the LRFD design of PPC I-beams and Bulb T-beams. The design procedure is presented first followed by two examples. The first is a simple span and the second is a two span. All Article and Equation references are to the LRFD code unless noted otherwise.

There are six standard beam cross sections supported by the Department. These are 36, 42, 48 and 54 in. PPC I-Beams and 63 and 72 in. Bulb T-Beams. Standard strand patterns for LRFD designs have been developed and are provided in Bridge Manual (BM) Section 3.4. Shear reinforcement has been standardized for most typical bridges and is shown in BM Section 3.4.5.2. All other reinforcement details (splitting steel, top flange reinforcement, bottom flange reinforcement, etc.) have been standardized and are shown on the base sheets. Aids for designing lifting loops are also shown in BM Section 3.4.

The main items a designer has to calculate are section properties, distribution factors, moment and shear envelopes, prestress losses, temporary stresses, service stresses and resisting moment capacities; all of which are used to determine the required strand pattern and concrete strength. A shear design is only required if the structure falls outside the limits of BM Table 3.4.5.2-1. For multi-span bridges, the longitudinal reinforcement in the deck over the piers must be designed.

For most cases the standard strand patterns should satisfy all the design requirements. However there could be some situations where a modification will need to be made, see BM Section 3.4.4.1 for permissible variations.

The sign convention used in the examples was noted by labeling negative results with “tension” and positive results with “compression” unless otherwise noted.

LRFD Design Procedure, Equations and Outline

Dead Loads, Section Properties and Distribution Factors

Calculate the dead loads per linear foot due to beam, deck, future wearing surface, parapets and any other dead loads on the bridge and group them in their appropriate types DC and DW. Calculate the section properties. See BM Figures (3.4.4.2-1 through 3.4.4.2-6). Live load distribution factors shall be calculated according to BM Section 3.3.1.

Modulus of Elasticity

The modulus of elasticity for concrete shall be determined from the following formulas:

$$E_{ci} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ci}} \qquad \text{(Eq. 5.4.2.4-1)}$$

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c}$$

Where:

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_c = modulus of elasticity of concrete (ksi)

K_1 = aggregate modification factor, taken as 1.0

w_c = unit weight of concrete, taken as 0.150 (kcf)

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)

f'_c = specified compressive strength of concrete for use in design (ksi)

Moment and Shear Envelopes

Calculate the moment and shear envelopes using software written for this task.

Strand Pattern Selection

The planning selection charts located in BM Section 2.3.6.1.3 can be used to determine a trial strand pattern. The properties of the trial strand pattern can be found in BM Tables 3.4.4.1-1 through 3.4.4.1-12.

Prestress Losses *(5.9.5)*

Calculate prestress losses.

Total Loss of Prestress (5.9.5.1)

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.5.1-1)

Where:

Δf_{pT} = total loss (ksi)

Δf_{pES} = loss in prestressing steel due to elastic shortening (ksi)

Δf_{pLT} = losses due to long term shrinkage and creep of concrete, and relaxation of the steel (ksi)

Instantaneous Losses (5.9.5.2)

It is the Department's policy to consider elastic losses only. Elastic gains are not considered.

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ (Eq. 5.9.5.2.3a-1)

In which:

$$f_{cgp} = \frac{F_t}{A} + \frac{F_t e^2}{I} - \frac{M_b(12)e}{I}$$

Assume F_t equals 90 percent of F_i for first iteration (C5.9.5.2.3a)

F_t = $0.9(F_i)$

F_i = $A_{ps}(f_{pbt})$

Calculate Δf_{pES} and verify assumption

$$\text{Assumption} \approx \frac{\left(f_{pbt} - \Delta f_{pES}\right)}{f_{pbt}}$$

Reiterate if necessary

Where:

Δf_{pES} = loss in prestressing steel due to elastic shortening (ksi)

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

f_{cgp} = concrete stress at the center of gravity of prestressing tendons due to prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (center) (ksi)

F_t = total prestressing force immediately after transfer (kips)

F_i = total prestressing force prior to transfer (kips)

e = eccentricity of centroid of strand pattern from NA of beam (in.)

M_b = bending moment due to beam self weight (kip-ft.)

A = area of beam (in.^{2})

I = moment of inertia of beam (in.^{4})

A_{ps} = total area of prestressing steel (in.^{2})

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

<u>Time Dependent Losses</u> (5.9.5.3)

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \quad \text{(Eq. 5.9.5.3-1)}$$

In which:

$$\gamma_h = 1.7 - 0.01H \quad \text{(Eq. 5.9.5.3-2)}$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})} \quad \text{(Eq. 5.9.5.3-3)}$$

Where:

Δf_{pLT} = losses due to long term shrinkage and creep of concrete, and relaxation of the steel (ksi)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

A_{ps} = total area of prestressing steel (in.^{2})

A = area of beam (in.^{2})

γ_h = correction factor for relative humidity

γ_{st} = correction factor for specified concrete strength at time of prestress transfer

Δf_{pR}= estimate of relaxation loss taken as 2.4 ksi for low relaxation strands (ksi)

H = relative humidity, assumed to be 70% in Illinois (%)

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)

Temporary Stresses *(5.9.4.1)*

Temporary stresses are checked immediately after the release of the strands when the concrete strength, f'_{ci}, is weakest. The force in the strands is taken to be the prestressing force immediately after transfer, F_t.

There are three support conditions to consider during this time frame. The first occurs when the strands are released and the beam is still setting on the prestressing bed. The second occurs when lifting the beam out of the prestressing bed. The third occurs when placing the beam in temporary storage at the fabrication plant. Theoretically, all three of these conditions could take place while the concrete is most vulnerable, however only the second condition will govern if current IDOT policies are followed. Therefore this is the only condition checked.

For this case, the stresses need to be checked in two locations. Those locations depend on whether a draped strand pattern or a straight strand pattern is used. For draped strand patterns, the stresses are checked at the center of the lifting loops and at the harping point. For straight strand patterns, the stresses are checked at the center of the lifting loops and at the center of the beam.

See Figure 1 for the support and loading diagram used to calculate the dead load moments for checking temporary stresses:

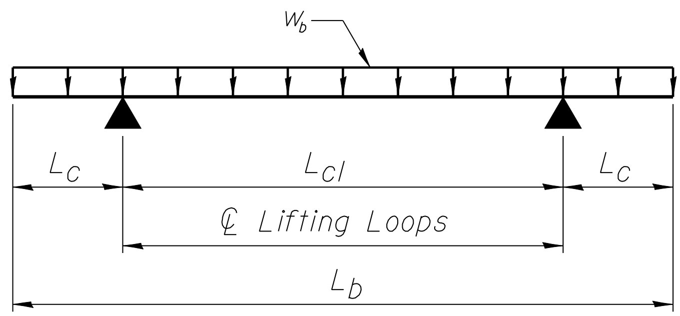


Figure 1

@ Lifting Loops:

$$M_{bts} = \frac{w_b (L_c)^2}{2}$$

@ Center:

$$M_{bts} = \frac{w_b (L_b)^2}{8} - \frac{w_b L_b L_c}{2}$$

@ Harping Point (0.4 L_b):

$$M_{bts} = \frac{3w_b (L_b)^2}{25} - \frac{w_b L_b L_c}{2}$$

Where:

M_{bts}= bending moment due to beam self weight with supports at temporary locations (kip-ft.)

w_b = weight per unit length of the beam (kip/ft.)

L_{cl} = length from center to center of lifting loops (ft.)

L_c = length of cantilever (ft.)

L_b = total length of beam (ft.)

Temporary Stress Limits for Concrete (5.9.4.1)

Compression:

$0.60f'_{ci}$

Tension:

$0.24\sqrt{f'_{ci}}$

Where:

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)

Calculate Temporary Stresses

@ Lifting Loops:

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} - \frac{M_{bts}(12)}{S_t}$$

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} + \frac{M_{bts}(12)}{S_b}$$

@ Center or Harping Point (0.4 L_b):

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} + \frac{M_{bts}(12)}{S_t}$$

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} - \frac{M_{bts}(12)}{S_b}$$

In which:

$F_t = A_{ps}(f_{pbt} - \Delta f_{pES})$

Where:

f_t = concrete stress at the top fiber of the beam (ksi)

f_b = concrete stress at the bottom fiber of the beam (ksi)

F_t = total prestressing force immediately after transfer (kips)

A = area of beam (in.^{2})

e = eccentricity of centroid of strand pattern from NA of beam (in.)

S_t = non-composite section modulus for the top fiber of the beam (in.^{3})

S_b = non-composite section modulus for the bottom fiber of the beam ($in.^3$)

M_{bts}= bending moment due to beam self weight with supports at temporary locations (kip-ft.)

A_{ps} = total area of prestressing steel ($in.^2$)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

Δf_{pES} = loss in prestressing steel due to elastic shortening (ksi)

Positive Moment Regions *(5.7.3)*

The design procedure for the positive moment area of a prestressed concrete member is outlined below. Please note that it is the Department's policy to not utilize non-prestressed tension reinforcement for positive moment.

Service Stresses (5.9.4.2)

Service limit state stresses are checked for the beam in its final placement in the structure. The concrete strength is equal to f'_c and the force in the strands is equal to F_S.

Compressive service stresses are calculated for the two applicable Service I load combinations given in Table 5.9.4.2.1-1. For simplicity, these have been given the nomenclature (a) and (b) in this design guide. Tensile service stresses are calculated for the one applicable Service III load combination given in Table 5.9.4.2.2-1. The factored Service I and Service III load combinations are found in Table 3.4.1-1 and the load factors have been applied to the equations shown below.

Service Stress Limits for Concrete after Losses (5.9.4.2)

Compression (For Service I load combination):

$0.60\phi_w f'_c$ (a)

$0.45f'_c$ (b)

Tension (For Service III load combination):

$0.19\sqrt{f'_c}$

Where:

f'_c = specified compressive strength of concrete for use in design (ksi)

ϕ_w = hollow column reduction factor, equals 1.0 for standard IDOT sections

Calculate Service Stresses

Service stresses are calculated from the following equations:

@ Center:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t^{'}} \quad \text{(a)}$$

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \quad \text{(b)}$$

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

In which:

$F_s = A_{ps}(f_{pbt} - \Delta f_{pT})$

Where:

f_t = concrete stress at the top fiber of the beam (ksi)

f_b = concrete stress at the bottom fiber of the beam (ksi)

f'_c = specified compressive strength of concrete for use in design (ksi)

F_S = total prestressing force after all losses (kips)

A = area of beam (in.^{2})

e = eccentricity of centroid of strand pattern from NA of beam (in.)

M_{DC1}= unfactored non-composite dead load moment of structural components and nonstructural attachments (kip-ft.)

M_{DC2}= unfactored composite dead load moment of structural components and nonstructural attachments (kip-ft.)

M_{DW1}= unfactored non-composite dead load moment of wearing surfaces and utilities (kip-ft.)

M_{DW2}= unfactored composite dead load moment of wearing surfaces and utilities (kip-ft.)

M_{LL+IM}=unfactored live load moment (HL-93) plus dynamic load allowance (kip-ft.)

S_t = non-composite section modulus for the top fiber of the beam (in.^{3})

S_b = non-composite section modulus for the bottom fiber of the beam (in.^{3})

S'_t = composite section modulus for the top fiber of the beam (in.^{3})

S'_b = composite section modulus for the bottom fiber of the beam (in.^{3})

A_{ps} = total area of prestressing steel (in.^{2})

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

Δf_{pT} = total loss (ksi)

<u>Fatigue Stresses</u> (5.5.3.1)

The compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed the limit shown below. The section properties used for calculating the compressive stress are determined based on whether the tensile stress limit shown below is exceeded. The tensile stress is calculated using the Fatigue I load combination plus effective prestress and permanent loads.

Fatigue Stress Limits for Concrete after Losses (5.5.3.1)

Compression:

$0.40f'_c$

Tension limit for determination of cracked verse uncracked section:

Uncracked $\leq 0.095\sqrt{f'_c} \leq$ Cracked

Where:

f'_c = specified compressive strength of concrete for use in design (ksi)

Calculate Fatigue Stresses

Fatigue stress is calculated from the following equation:

@ Center:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S'_t}\right] + 1.5\frac{M_{FL+IM}(12)}{S'_t}$$

Tension stress is calculated from the following equation:

(This stress is only used to determine whether the section is considered cracked or uncracked for fatigue evaluation)

@ Center:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (1.5)\frac{M_{FL+IM}(12)}{S_b^{'}}$$

Where:

- f_t = concrete stress at the top fiber of the beam (ksi)
- f_b = concrete stress at the bottom fiber of the beam (ksi)
- f'_c = specified compressive strength of concrete for use in design (ksi)
- F_S = total prestressing force after all losses (kips)
- A = area of beam (in.^{2})
- e = eccentricity of centroid of strand pattern from NA of beam (in.)
- M_{DC1}= unfactored non-composite dead load moment of structural components and nonstructural attachments (kip-ft.)
- M_{DC2}= unfactored composite dead load moment of structural components and nonstructural attachments (kip-ft.)
- M_{DW1}= unfactored non-composite dead load moment of wearing surfaces and utilities (kip-ft.)
- M_{DW2}= unfactored composite dead load moment of wearing surfaces and utilities (kip-ft.)
- M_{FL+IM}=unfactored fatigue live load moment plus dynamic load allowance (kip-ft.)
- S_t = non-composite section modulus for the top fiber of the beam (in.^{3})
- S_b = non-composite section modulus for the bottom fiber of the beam (in.^{3})
- S'_t = composite section modulus for the top fiber of the beam (in.^{3})
- S'_b = composite section modulus for the bottom fiber of the beam (in.^{3})

<u>Strength I Moment</u>

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$ (Table 3.4.1-1)

Impact shall be taken as 33% (Table 3.6.2.1-1). Engineering judgment may be used when determining the value of the "η" load modifiers specified in Article 1.3.2. As these are normally assumed to be 1.0 in standard bridges and therefore do not affect the design, they will not be addressed any further in this design guide.

<u>Factored Flexural Resistance</u>

$M_r = \phi M_n$

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right)\left(\frac{1}{12}\right)$$ rectangular (Eq. 5.7.3.2.2-1)

$$M_n = \left[A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)\right]\left(\frac{1}{12}\right)$$ flanged (Eq. 5.7.3.2.2-1)

In which:

$a = \beta_1 c$

$$c = \frac{A_{ps}f_{pu}}{0.85f'_c\beta_1 b + kA_{ps}\frac{f_{pu}}{d_p}}$$ rectangular (Eq. 5.7.3.1.1-4)

$$c = \frac{A_{ps}f_{pu} - 0.85f'_c(b - b_w)h_f}{0.85f'_c\beta_1 b_w + kA_{ps}\frac{f_{pu}}{d_p}}$$ flanged (Eq. 5.7.3.1.1-3)

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ If $f_{pe} \geq 0.5 f_{pu}$ (Eq. 5.7.3.1.1-1)

$f_{pe} = f_{pu} - \Delta f_{pT}$

$$\phi = 0.75 \leq 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \leq 1.0$$ (Eq. 5.5.4.2.1-1)

$$\beta_1 = 0.65 \leq 0.85 - 0.05\left(f'_c - 4.0\right) \leq 0.85$$ (5.7.2.2)

Where:

M_r = factored flexural resistance of a section in bending (kip-ft.)

M_n = nominal flexural resistance (kip-ft.)

a = depth of equivalent rectangular stress block (in.)

c = distance from the extreme compression fiber to the neutral axis (in.)

f_{ps} = average stress in prestressing steel at nominal bending resistance (ksi)

ϕ = resistance factor (maximum reinforcement provisions were deleted in the 2005 LRFD code and are now checked indirectly using Eq. 5.5.4.2.1-1)

β_1 = stress block factor

A_{ps} = total area of prestressing steel (in.^{2})

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)

b = width of the compression face of the member (equals effective flange width) (in.)

b_w = web width (equals width of the top flange of the beam) (in.)

h_f = compression flange depth (equals deck thickness) (in.)

f_{pu} = specified tensile strength of prestressing steel (ksi)

f_{pe} = effective stress in the prestressing steel after losses (ksi)

k = 0.28 (Table C5.7.3.1.1-1)

f_c' = specified compressive strength of concrete for use in design (ksi)

d_t = distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.)

Δf_{pT} = total loss (ksi)

The factored flexural resistance equations shown above have been simplified to include only the prestressing steel. All other reinforcement shall be ignored.

Minimum Reinforcement

The Department requires minimum reinforcement be adequate to develop a factored flexural resistance of at least the cracking moment for prestressed beams. This is done to ensure ductility in the event of an unexpected overload. This requirement may be waived if the factored ultimate strength of the beam exceeds 1.33 times the applied load.

$M_r \geq M_{cr}$ (5.7.3.3.2)

In which:

$$M_{cr} = \gamma_3\left[\frac{S'_b(\gamma_1 f_r + \gamma_2 f_{cpe})}{12} - M_{DC1}\left(\frac{S'_b}{S_b} - 1\right)\right]$$ (Eq. 5.7.3.3.2-1)

$$f_r = 0.24\sqrt{f'_c}$$ (5.4.2.6)

$$f_{cpe} = \frac{F_s}{A} + \frac{F_s e}{S_b}$$

Where:

M_u = factored moment at the section (kip-ft.)

M_r = factored flexural resistance of a section in bending (kip-ft.)

M_{cr}= cracking moment (kip-ft.)

f_r = modulus of rupture of concrete (ksi)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

S_b = non-composite section modulus for the bottom fiber of the beam (in.^{3})

S'_b = composite section modulus for the bottom fiber of the beam (in.^{3})

M_{DC1}= unfactored non-composite dead load moment of structural components and nonstructural attachments (kip-ft.)

f'_c = specified compressive strength of concrete for use in design (ksi)

F_S = total prestressing force after all losses (kips)

A = area of beam (in.^{2})

e = eccentricity of centroid of strand pattern from NA of beam (in.)

γ_1 = flexural cracking variability factor

= 1.6 for non-segmentally constructed members

γ_2 = prestress variability factor

= 1.1 for bonded tendons

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement

= 1.00 for prestressed concrete structures

Negative Moment Regions *(5.7.3)*

Precast prestressed concrete members made continuous at interior supports shall be proportioned to resist the total moment due to composite dead loads and live loads including dynamic load allowance. The negative moment capacity shall be based on the 28 day compressive strength of the concrete in the girders regardless of the diaphragm concrete strength (see Article 5.14.1.4.10). Both the top and bottom longitudinal reinforcement in the deck may be utilized to resist this moment. The typical longitudinal reinforcement configuration including size and spacing limitations is shown in Figure 2.

An example design procedure could consist of the following steps:

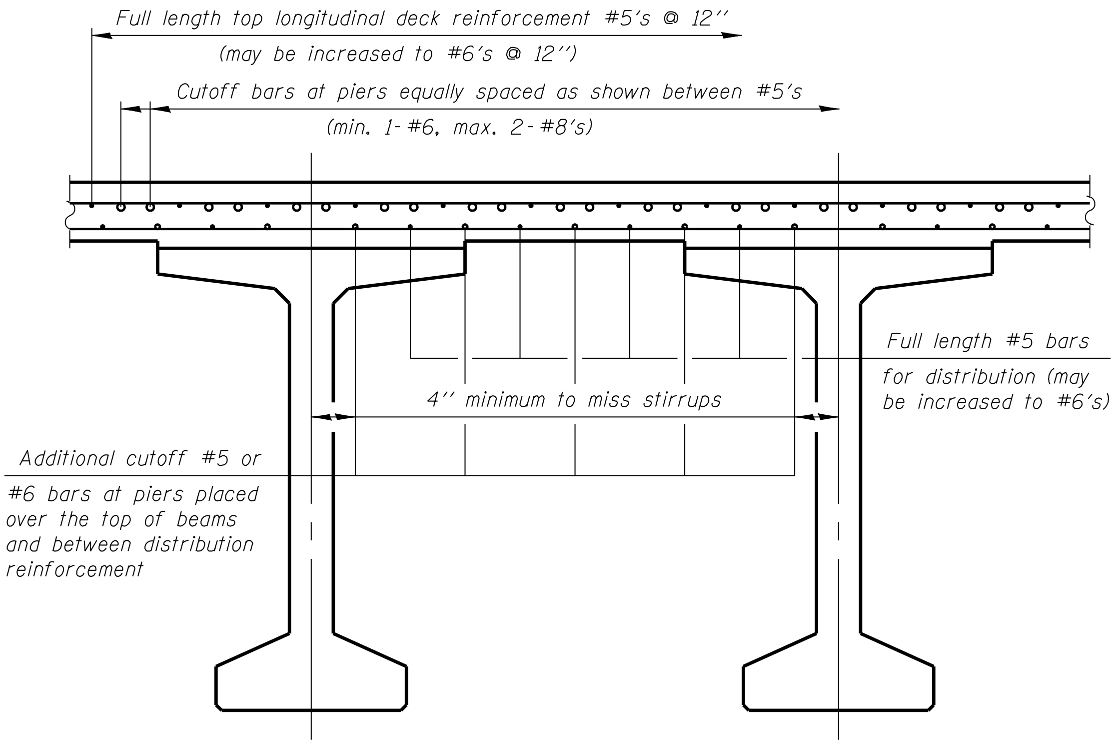
1. Calculate the maximum moments at the pier.
2. Estimate the total area of reinforcement required.
3. Estimate the area of reinforcement required to continue past the cutoff point.
4. Determine trial reinforcement arrangement to satisfy number 2 and number 3 using the limitations defined in Figure 2.
5. Check the adequacy of the total reinforcement and the continuing reinforcement for the strength limit state, the service limit state and the fatigue limit state.

The theoretical cutoff point for the longitudinal reinforcement shall be located where the resisting moment capacity of the continuing reinforcement is greater than or equal to the applied moment at that location. However this point shall not be less than a distance of 0.2L from either side of the pier (where L is the length of span under consideration). The cutoff reinforcement shall be fully developed. The actual cutoff point for the reinforcement is then determined by adding an embedment length past the theoretical cutoff point per Article 5.11.1.2.1.

The area of continuing reinforcement is determined from Article 5.11.1.2.3. It states that at least one third of the total tension reinforcement provided shall extend an embedment length past the point of inflection. The Department uses the one third rule to calculate the area of continuing reinforcement required however this reinforcement is not terminated at the inflection point. Instead this reinforcement is extended the full length of the deck. This was done mainly for simplicity and since the amount of reinforcement saved is negligible.

The compressive stress in the bottom fiber of the bottom flange shall be checked near the pier. The compression block area at the bottom flange of the beam has been modified as shown in Figure 3 in order to simplify the calculations. The tapered portion of the bottom flange has been converted into a rectangle that is one third the height of the triangle it replaced.

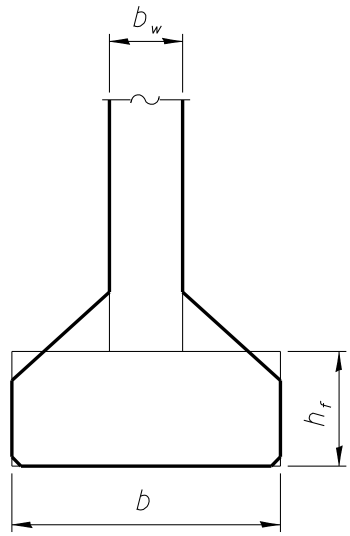
The provisions of Article 5.14.1.4 address design requirements for positive moments which may occur in the negative moment regions of simple span prestressed beams made continuous for live load and superimposed dead loads. These positive moments can be caused by creep and shrinkage in the girders and deck slabs and/or live loads from remote spans. Experience has demonstrated that the Department's continuity details (2-# 8 G6 bars for I-Beams and 3-# 8 G6 bars for Bulb T-Beams) have been fairly successful in minimizing distress from these forces. However, fabrication issues have prompted the Department to change the detailing of the G6 bar to a spliced bar assembly shown on the beam base sheets. The new dimensions address some of the provision's concerns about development of this reinforcement. The Department believes that the intent of Article 5.14.1.4 has been sufficiently addressed with the new G6 rebar revisions and no further design consideration is required for structures within the Department's design parameters and details. Structures beyond the Department's design parameters and details are subject to all requirements of Article 5.14.1.4.



CROSS SECTION AT PIER

(minimum spacing of all reinforcement shall be 4'')

Figure 2



Beam	b	b_w	h_f
36'' I-Beam	18''	6''	8''
42'' I-Beam	22''	6''	8.6''
48'' I-Beam	22''	7.5''	9.4''
54'' I-Beam	22''	6''	9.4''
63'' Bulb-T	26''	6''	7.5''
72'' Bulb-T	26''	6''	7.5''

SIMPLIFIED COMPRESSION BLOCK

Figure 3

Strength I Moment

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$ (Table 3.4.1-1)

Impact shall be taken as 33% (Table 3.6.2.1-1). Engineering judgment may be used when determining the value of the "η" load modifiers specified in Article 1.3.2. As these are normally assumed to be 1.0 in standard bridges and therefore do not affect the design, they will not be addressed any further in this design guide.

Factored Flexural Resistance

$M_r = \phi M_n$

$$M_n = A_s f_s\left(d_s - \frac{a}{2}\right)\left(\frac{1}{12}\right)$$ rectangular (Eq. 5.7.3.2.2-1)

$$M_n = \left[A_s f_s\left(d_s - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)\right]\left(\frac{1}{12}\right)$$ flanged (Eq. 5.7.3.2.2-1)

In which:

$a = \beta_1 c$

$$c = \frac{A_s f_s}{0.85 f'_c \beta_1 b}$$ rectangular (Eq. 5.7.3.1.1-4)

$$c = \frac{A_s f_s - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w}$$ flanged (Eq. 5.7.3.1.1-3)

$f_s = f_y$ if $\frac{c}{d_s} \le 0.6$ (5.7.2.1)

if $\frac{c}{d_s} > 0.6$, f_s shall be determined using strain compatibility

$$\phi = 0.75 \le 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \le 0.9$$ (Eq. 5.5.4.2.1-2)

$$\beta_1 = 0.65 \le 0.85 - 0.05\left(f'_c - 4.0\right) \le 0.85$$ (5.7.2.2)

Where:

M_u = factored moment at the section (kip-ft.)

M_r = factored flexural resistance of a section in bending (kip-ft.)

M_n = nominal flexural resistance (kip-ft.)

a = depth of equivalent rectangular stress block (in.)

c = distance from the extreme compression fiber to the neutral axis (in.)

f_s = stress in the non-prestressed tensile reinforcement at nominal flexural resistance (ksi)

ϕ = resistance factor (maximum reinforcement provisions were deleted in the 2005 LRFD code and are now checked indirectly using Eq. 5.5.4.2.1-2)

β_1 = stress block factor

A_s= total area of non prestressed tensile reinforcement within the effective flange width (in.^{2})

d_s = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (in.)

b = width of the compression face of the member (equals bottom flange width) (in.)

b_w = web width (equals width of web of the beam) (in.)

h_f = compression flange depth (equals bottom flange thickness, see Figure 3) (in.)

f_y = specified minimum yield strength of non-prestressed tensile reinforcement (ksi)

f'_c = specified compressive strength of concrete for use in design (ksi)

d_t = distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.)

The factored flexural resistance equations shown above have been simplified to include only the tension reinforcement. All other reinforcement shall be ignored.

Minimum Reinforcement

The Department requires minimum reinforcement be adequate to develop a factored flexural resistance of at least the cracking moment for prestressed beams. This is done to ensure ductility in the event of an unexpected overload. This requirement may be waived if the factored ultimate strength of the beam exceeds 1.33 times the applied load.

$$M_r \geq M_{cr} \qquad (5.7.3.3.2)$$

In which: Since M_{dnc} and f_{cpe} both equal zero,

$$M_{cr} = \gamma_3\gamma_1 \frac{S'_{ts}(f_r)}{12} \qquad \text{(Eq. 5.7.3.3.2-1)}$$

$f_r = 0.24\sqrt{f'_c}$ (5.4.2.6)

Where:

M_r = factored flexural resistance of a section in bending (kip-ft.)

M_{cr} = cracking moment (kip-ft.)

S'_{ts} = composite section modulus for the top fiber of the slab ($in.^3$)

f_r = modulus of rupture of concrete (ksi)

f'_c = specified compressive strength of concrete for use in design (ksi)

γ_1 = flexural cracking variability factor

= 1.6 for non-segmentally constructed members

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement

= 0.75 for ASTM A706 reinforcement

Calculation of Stresses for Service and Fatigue Limit States

For the calculation of stresses for service and fatigue limit states, the straight line theory of stress and strain shall apply. See Article 5.7.1 and Figure 4 below.

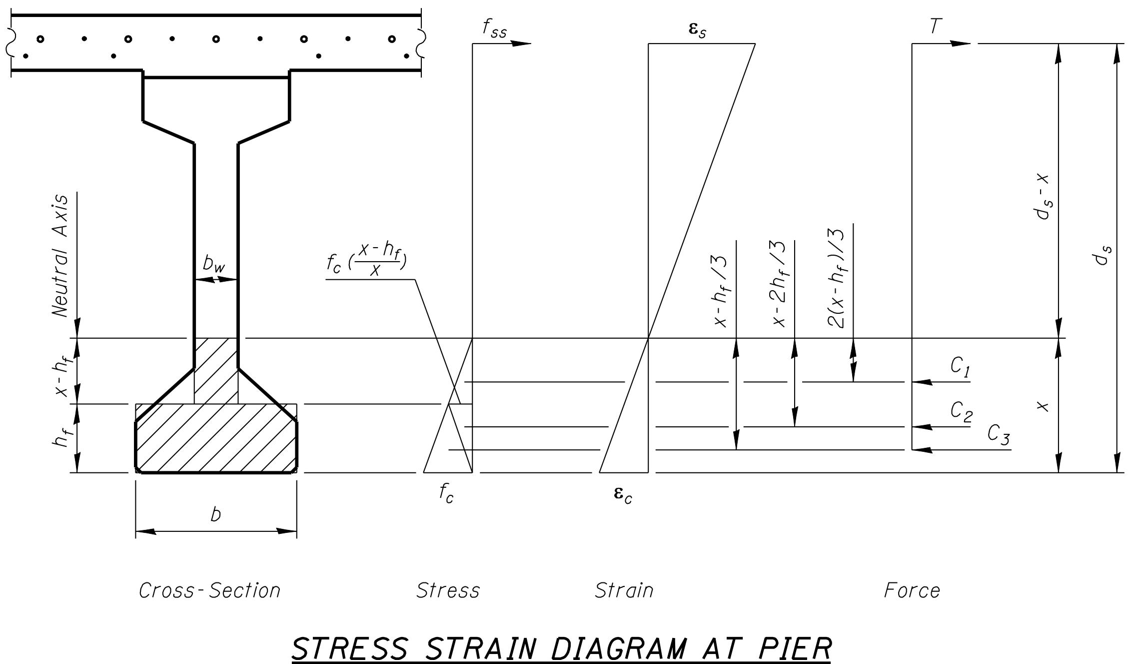


Figure 4

Determine expression for f_{ss}:

$$f_{ss} = \varepsilon_s E_s$$

$$f_c = \varepsilon_c E_c \qquad \text{solve for } \varepsilon_c = \frac{f_c}{E_c}$$

$$n = \frac{E_s}{E_c}$$

$$\frac{\varepsilon_s}{\varepsilon_c} = \frac{d_s - x}{x} \qquad \text{solve for } \varepsilon_s = \varepsilon_c\left(\frac{d_s - x}{x}\right)$$

Substitute $\varepsilon_c = \dfrac{f_c}{E_c}$ into $\varepsilon_s = \varepsilon_c\left(\dfrac{d_s - x}{x}\right)$

$$\varepsilon_s = \frac{f_c}{E_c}\left(\frac{d_s - x}{x}\right)$$

Substitute $\varepsilon_s = \dfrac{f_c}{E_c}\left(\dfrac{d_s - x}{x}\right)$ into $f_{ss} = \varepsilon_s E_s$

$$f_{ss} = f_c \frac{E_s}{E_c}\left(\frac{d_s - x}{x}\right)$$

Substitute $n = \dfrac{E_s}{E_c}$ into $f_{ss} = f_c \dfrac{E_s}{E_c}\left(\dfrac{d_s - x}{x}\right)$

$$f_{ss} = f_c n\left(\frac{d_s - x}{x}\right)$$

Determine expressions for x and S_{bc}:

By equilibrium:

Case 1: $T = C_1 + C_2 + C_3$

Case 2: $\sum M = 0$ Summation of moments about neutral axis

(Note Case 1 is used to determine x and Case 2 is used to determine S_{bc})

Where:

$$T = A_s f_{ss} = A_s f_c n\left(\frac{d_s - x}{x}\right)$$

$$C_1 = f_c\left(\frac{x-h_f}{x}\right)\left(\frac{x-h_f}{2}\right)b_w$$

$$C_2 = f_c\left(\frac{x-h_f}{x}\right)\left(\frac{h_f}{2}\right)b$$

$$C_3 = f_c\left(\frac{h_f}{2}\right)b$$

Case 1: $A_s f_c n\left(\frac{d_s-x}{x}\right) = f_c\left(\frac{x-h_f}{x}\right)\left(\frac{x-h_f}{2}\right)b_w + f_c\left(\frac{x-h_f}{x}\right)\left(\frac{h_f}{2}\right)b + f_c\left(\frac{h_f}{2}\right)b$

Divide by f_c

$$A_s n\left(\frac{d_s-x}{x}\right) = \left(\frac{x-h_f}{x}\right)\left(\frac{x-h_f}{2}\right)b_w + \left(\frac{x-h_f}{x}\right)\left(\frac{h_f}{2}\right)b + \left(\frac{h_f}{2}\right)b$$

Multiply by x

$$A_s n(d_s-x) = (x-h_f)\left(\frac{x-h_f}{2}\right)b_w + (x-h_f)\left(\frac{h_f}{2}\right)b + \left(\frac{h_f}{2}\right)bx$$

Insert known variables, put into quadratic form and solve for x

Case 2: $M = A_s f_c n\left(\frac{d_s-x}{x}\right)(d_s-x) + f_c\left(\frac{x-h_f}{x}\right)\left(\frac{x-h_f}{2}\right)b_w\left(\frac{2(x-h_f)}{3}\right) + f_c\left(\frac{x-h_f}{x}\right)\left(\frac{h_f}{2}\right)b\left(x-\frac{2h_f}{3}\right) + f_c\left(\frac{h_f}{2}\right)b\left(x-\frac{h_f}{3}\right)$

Divide by f_c

$$\frac{M}{f_c} = A_s n\left(\frac{d_s-x}{x}\right)(d_s-x) + \left(\frac{x-h_f}{x}\right)\left(\frac{x-h_f}{2}\right)b_w\left(\frac{2(x-h_f)}{3}\right) + \left(\frac{x-h_f}{x}\right)\left(\frac{h_f}{2}\right)b\left(x-\frac{2h_f}{3}\right) + \left(\frac{h_f}{2}\right)b\left(x-\frac{h_f}{3}\right)$$

$S_{bc} = \frac{M}{f_c}$ Therefore S_{bc} equals above expression

<u>Control of Cracking by Distribution of Reinforcement</u> (5.7.3.4)

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.7.3.4-1)}$$

In which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

- s = spacing of non-prestressed tensile reinforcement (equals spacing of top row of longitudinal reinforcement) (in.)
- β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
- γ_e = exposure factor (use 0.75 for Class 2 exposure)
- d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)
- f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)
- h = overall thickness or depth of the component (equals total depth of beam and slab) (in.)

<u>Fatigue of Reinforcement</u> (5.5.3.2)

$$\gamma(\Delta f) \leq (\Delta F)_{TH} \qquad \text{(Eq. 5.5.3.1-1)}$$

In which:

$$(\Delta F)_{TH} = 24 - 0.33 f_{min} \qquad \text{(Eq. 5.5.3.2-1)}$$

Where:

- $(\Delta F)_{TH}$ = constant-amplitude fatigue threshold, as specified in Article 5.5.3.2 (ksi)
- Δf = force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)
- f_{min} = minimum live-load stress resulting from the Fatigue I load combination plus permanent loads (ksi) (positive if tension, negative if compression)
- γ = load factor specified in Table 3.4.1-1 for Fatigue I load combination

<u>Compressive Stress Check Near Pier</u>

The compressive stress in the bottom flange is checked a transfer length from the end of the beam near the pier for both the service and fatigue limit states. The transfer length is 60 strand diameters as defined in Article 5.11.4.1. This location is checked because it is the first point where the prestressing force is fully effective. The standard strand patterns for multiple spans have been developed to account for this stress however, due to the large number of loading cases, it may be necessary to drape additional strands. For fabrication purposes the maximum number of draped strands shall be 16.

Compressive service stresses are calculated for the two applicable Service I load combinations given in Table 5.9.4.2.1-1. For simplicity, these have been given the nomenclature (a) and (b) in this design guide. The factored Service I load combination definitions are found in Table 3.4.1-1 and the factors have been applied to the equations shown below.

Compressive fatigue stress is calculated for the Fatigue I load combination plus one-half the sum of effective prestress and permanent loads as shown in Article 5.5.3.1. The factored Fatigue I load combination definitions are found in Table 3.4.1-1 and the factors have been applied to the equations shown below.

It should be noted that both the service and fatigue stresses are conservatively calculated based on a cracked section. In addition the stress resulting from the eccentricity due to the prestressing force is also calculated based on a cracked section. This was done to account for the redistributuion of this stress once the section cracks.

Service stress limits for concrete after losses:

The service stress limits for concrete after losses are the same as those shown on page 3.4-8 of this guide.

Calculate service stresses:

Service stresses are calculated from the following equations:

@ Transfer point from pier:

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_{bc}} \qquad \text{(a)}$$

$$f_b \quad = \quad \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}} \qquad \text{(b)}$$

Where:

F_S = total prestressing force after all losses (kips)

A = area of beam (in.^{2})

e = eccentricity of centroid of strand pattern from NA of beam (in.)

S_{bc} = composite cracked section modulus for the bottom fiber of the beam (in.^{3})

Fatigue stress limits for concrete after losses:

The fatigue stress limits for concrete after losses are the same as those shown on page 3.4-10 of this guide. However the tensile stress limit which is used to determine whether the section is cracked or uncracked need not be checked since the section is conservatively assumed to be cracked.

Calculate Fatigue Stresses

Fatigue stress is calculated from the following equation:

@ Transfer point from pier:

$$f_b \quad = \quad 0.5\left[\frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_{bc}}$$

Where:

f_b = concrete stress at the bottom fiber of the beam (ksi)

f'_c = specified compressive strength of concrete for use in design (ksi)

F_S = total prestressing force after all losses (kips)

A = area of beam (in.^{2})

e = eccentricity of centroid of strand pattern from NA of beam (in.)

M_{DC1}= unfactored non-composite dead load moment of structural components and nonstructural attachments (kip-ft.)

M_{DC2}= unfactored composite dead load moment of structural components and nonstructural attachments (kip-ft.)

M_{DW1}= unfactored non-composite dead load moment of wearing surfaces and utilities (kip-ft.)

M_{DW2}= unfactored composite dead load moment of wearing surfaces and utilities (kip-ft.)

M_{FL+IM}=unfactored fatigue live load moment plus dynamic load allowance (kip-ft.)

S_{bc} = composite cracked section modulus for the bottom fiber of the beam (in.^{3})

Calculate Camber and Deflection

Camber, which is the result of the difference between the upward deflection caused by the prestressing forces and the downward deflection due to the weight of the beam and slab, must be considered when determining the seat elevations. The top of the beam shall be set to provide a minimum positive fillet height of 0.5 inch above any point on the beam.

Camber will vary with the age of the member, primarily because of two factors; loss of prestress which will tend to decrease the deflection, and creep which will tend to increase the deflection. Because of this, correction factors are used in the equations for calculating camber. Factors of 1.80 and 1.85 are used on the upward deflection caused by the prestressing force and downward deflection due to member weight, respectively. These factors are based on the PCI Design Handbook for the time at erection and have been incorporated into the equations shown below.

Initial Resultant Camber

$$\text{Camber} = D_{cp} - D_{cb}$$

In which:

$$D_{cp} = \frac{F_t(12L)^2 e}{8E_{ci} I}(1.80) \quad \text{for straight strand patterns}$$

$$D_{cp} = \frac{F_t(12L)^2}{E_{ci} I}\left[0.0983\, e_{center} + 0.0267\, e_{end}\right](1.80) \quad \text{for draped strand patterns}$$

$$D_{cb} = \frac{5 w_b (12L)^4}{384(12) E_{ci} I}(1.85)$$

Where:

D_{cp} = upward deflection due to prestressing (in.)

D_{cb} = downward deflection due to beam weight (in.)

F_t = total prestressing force immediately after transfer (kips)

L = span length (ft.)

e = eccentricity of centroid of strand pattern from NA of beam (in.)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

I = moment of inertia of beam (in.^{4})

w_b = weight per unit length of the beam (kip/ft.)

Final Resultant Camber for Computing Bearing Seat Elevations

The dead loads to be considered for adjusting the grade line are those which will appreciably increase the downward deflection of the beams after they have been erected. This load is the weight of the slab. The weight of future wearing surface is not included.

Normally, the deflection caused by the weight of the curbs, parapets and handrails is insignificant and can be disregarded. In cases where they might appear significant, the above dead loads should be included when adjusting the grade line for dead load deflections.

$$\text{Camber} = D_{cp} - D_{cb} - D_{cs}$$

In which:

$$D_{cs} = \frac{5 w_s (12L)^4}{384(12) E_c I}$$

Where:

D_{cp} = upward deflection due to prestressing (in.)

D_{cb} = downward deflection due to beam weight (in.)

D_{cs} = downward deflection due to slab (in.)

w_s = weight per unit length of the slab (kip/ft.)

L = span length (ft.)

E_c = modulus of elasticity of concrete (ksi)

I = moment of inertia of beam (in.^{4})

<u>Downward Deflections Due to Slab Weight for Adjusting Grade Elevations</u>

@0.25 point = $0.7125D_{cs}$

@0.50 point = D_{cs}

@0.75 point = $0.7125D_{cs}$

Shear Design (5.8)

The standard transverse (shear) reinforcement patterns developed by the Department (Bridge Manual Section 3.4.5.2) to be used away from the ends of beams are applicable to most typical bridges designed and constructed in Illinois. However, when the beam length or spacing falls outside of the limits given in Table 3.4.5.2-1 of the Bridge Manual, transverse reinforcement is required to be designed. As discussed in Section 3.4.5.1 of the Bridge Manual, there are several options available in the LRFD Code for the design of transverse reinforcement (shear stirrups) in prestressed beams.

At the end regions of beams, special standard splitting steel details are required. These are discussed in Section 3.4.8 of the Bridge Manual and details are provided on Departmental base sheets.

A simplified and conservative approach to the general procedure of the sectional model (LRFD Article 5.8.3.4.2) may be used when transverse reinforcement away from beam ends is required to be designed. Rigorous application of the design procedure outlined in LRFD Article 5.8.3.4.2 is not prohibited by the Department, but is not necessary. A descriptive outline with commentary (in italics) for LRFD transverse reinforcement design developed by the BBS is presented below. Detailed calculations are also given in the presented examples of this design guide. Several conservative simplifications are allowed and described over the course of the design guide. These simplifications reduce computational intensity. The attached examples, however, show full calculations without these simplifications.

Final spacing of transverse reinforcement shall be checked at the critical sections for shear as well as at the tenth points along the beam or span. Final spacing for reinforcement shall satisfy the following requirements:

Nominal Shear Resistance	(5.8.3.3)
Maximum Spacing of Transverse Reinforcement	(5.8.2.7)
Minimum Transverse Reinforcement	(5.8.2.5)
Interface Shear Transfer	(5.8.4)

In addition, longitudinal reinforcement in beams shall be checked near supports according to Article 5.8.3.5.

Location of Critical Section *(5.8.3.2)*

The location of the critical section is taken as d_v from the face of the support, where d_v is the effective shear depth as calculated below. If harped strands are present, d_v will change along the length of the beam, making the d_v calculations iterative. For this reason, the location of the critical section for shear may be taken as 0.72h from the face of the support in lieu of more complicated computations. The face of the support is defined as the inside face of the concrete diaphragm for fixed abutments and piers, and the inside edge of the bearing for expansion abutments and piers.

Nominal Shear Resistance *(5.8.3.3)*

The factored shear resistance, V_r, shall be taken as:

$$V_r = \phi V_n \qquad \text{(Eq. 5.8.2.1-2)}$$

Where:

ϕ = Resistance factor for shear as specified in Article 5.5.4.2
= 0.9

V_n = Nominal shear resistance as defined below (k)

The nominal shear resistance, V_n, shall de determined as the lesser of:

$V_n = V_c + V_s + V_p$ (Eq. 5.8.3.3-1)

$V_n = 0.25f'_c b_v d_v + V_p$ (Eq. 5.8.3.3-2)

Where:

V_c = Shear resistance due to concrete (k)

$= 0.0316\beta\sqrt{f'_c}\,b_v d_v$ (Eq. 5.8.3.3-3)

V_s = Shear resistance due to transverse reinforcing steel (k)

$= \frac{A_v f_y d_v \cot\theta}{s}$ (Eq. C5.8.3.3-1)

V_p = Vertical component of prestressing force in direction of applied shear (k). PCI Bridge Design Manual defines this as the force in the vertical direction due to prestressing, not the force in the direction of the angle of inclination, Θ.

The parameters of V_c and V_s are as defined later in this design guide.

Solving Equations 5.8.3.3-1, 5.8.3.3-3, C5.8.3.3-1, and 5.8.2.1-2 for s and setting V_r equal to V_u gives a maximum spacing of:

$$s = \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$ (variables defined later)

As Equation 5.8.3.3-2 typically does not control the design, design for Equation 5.8.3.3-1 first, then check Equation 5.8.3.3-2 when a final design is reached.

Effective Shear Depth d_v *(5.8.2.9)*

The effective shear depth, d_v, is taken as:

$d_v = d_e - \frac{a}{2}$ (C5.8.2.9)

Where:

d_e = effective depth from top of slab to centroid of prestressing strands (positive moment regions) or depth from bottom of beam to centroid of longitudinal deck reinforcement (negative moment regions), at the section under consideration (in.)

a = depth of equivalent stress block (in.), taken as $a = c\beta_1$. See moment design section for computation of c.

d_v need not be taken as less than the greater of $0.9d_e$ or 0.72h.

The effective shear depth is taken as the distance between the centroid of the equivalent compression block and the centroid of the tension reinforcement at the section under consideration. The procedure for determining this may become tedious, especially when harped strands are used and the centroid of the tension reinforcement changes along the length of the beam. To avoid computational intensity, d_v may be taken as 0.72h, where h is the depth of the composite section. This will yield more conservative results than performing full calculations.

Vertical Component of Prestressing Force V_p

V_p, the vertical component of the prestressing force, may be taken as:

$$V_p = A_{ps}^{harped} f_{px} \sin\Psi$$

Where:

A_{ps}^{harped} = Area of harped strands at section under consideration ($in.^2$). At sections between and including harping points, A_{ps}^{harped} is taken as zero.

f_{px} = Design stress in pretensioned strand at nominal flexural strength at section under consideration (ksi). f_{px} varies linearly from zero to f_{pe} from the end of the beam to the end of the transfer length of the strands. It varies linearly again from f_{pe} to f_{ps} from the end of the transfer length of the strands to the point where the strands are fully developed. At points where the strands are fully developed, $f_{px} = f_{ps}$. Variable definitions and equations for calculating f_{px}, f_{pe}, f_{ps}, development lengths, and transfer lengths are given below. For a graphical depiction of f_{px}, see Fig. C5.11.4.2-1.

Ψ = Angle of harped strands from horizontal (degrees)

Determine Transfer Lengths, Development Lengths, and f_{px} (5.11.4)

The transfer length may be taken as 60 strand diameters from the end of the beam (5.11.4.1).

The development length, l_d (in.), may be found using the following equation for positive moment regions. For negative moment regions, the development length for positive moment regions may be used.

$$l_d \geq K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \qquad \text{(Eq. 5.11.4.2-1)}$$

Where:

K = Variable to account for differences in strand fabrication, taken as 1.6 for members with a depth greater than 24 inches.

f_{pe} = Effective stress in prestressing steel after losses (ksi). See moment design for calculation of losses.

d_b = Diameter of prestressing steel strand (in.)

f_{ps} = Average stress in prestressing steel (ksi)

$$= f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(Eq. 5.7.3.1.1-1)}$$

Where:

f_{pu} = Specified tensile strength of prestressing steel (ksi)

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{(Eq. 5.7.3.1.1-2)}$$

Where f_{py} is the yield strength of the prestressing steel, taken as $0.9f_{pu}$ (Table 5.4.4.1-1). k = 0.28 for low relaxation strands.

c = Distance from extreme compression fiber to neutral axis (in.). See moment design for computation of c.

d_p = Distance from extreme compression fiber to centroid of tension reinforcement (in.).

For sections between the end of the beam and the end of the transfer length:

$$f_{px} = \frac{f_{pe} l_{px}}{60 d_b} \qquad \text{(Eq. 5.11.4.2-2)}$$

For sections between the end of the transfer length and the end of the development length, the following equation may be used to calculate f_{px}. Note that l_d and $60d_b$ are taken from the end of the beam in this equation. If CL bearing is being used as a reference point instead of the end of the beam, the equation should be modified accordingly.

$$f_{px} = f_{pe} + \frac{l_{px} - 60d_b}{l_d - 60d_b}\left(f_{ps} - f_{pe}\right) \qquad \text{(Eq. 5.11.4.2-3)}$$

Where:

- f_{ps} = Average stress in prestressing steel (ksi), defined as above.
- f_{pe} = Effective stress in prestressing steel after losses (ksi). See moment design for calculation of losses.
- l_{px} = Length from end of beam to section under consideration (in.)
- d_b = Diameter of prestressing steel strand (in.)

Calculation of V_p may be time-consuming for sections near the ends of beams. In lieu of calculations, V_p may be conservatively taken as zero for the entire length of the beam. However, for beams with many harped strands, V_p may become significant, and taking it as zero may be overly conservative.

Shear Resistance Due to Concrete V_c

The shear resistance due to concrete, V_c, is taken as:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \qquad \text{(Eq. 5.8.3.3-3)}$$

Where:

f'_c = Compressive strength of concrete beam (ksi)

d_v = Effective shear depth, as defined above (in.)

b_v = Width of web of member (in.)

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear

= $\frac{4.8}{1+750\varepsilon_s}$ for all sections containing at least the minimum transverse reinforcement specified in Article 5.8.2.5 (Eq. 5.8.3.4.2-1). Assume that the requirements of Article 5.8.2.5 are satisfied, then check later in design.

Where:

ε_s = longitudinal strain (in./in.)

$$= \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{\left(E_sA_s + E_pA_{ps}\right)} \geq 0 \qquad \text{(Eq. 5.8.3.4.2-4)}$$

Where:

$|M_u|$ = Absolute factored Strength I moment (k-in.), not to be taken as less than $|V_u - V_p|d_v$

d_v = Effective shear depth as defined above (in.)

N_u = Axial force in beam, taken as zero (k)

V_u = Factored Strength I shear (k)

V_p = Vertical component of prestressing force as defined above (k)

A_{ps} = Area of prestressing steel on flexural tension side of member (in.^{2}). Fig. 5.8.3.4.2-1 allows the flexural tension side of the member to be found using 0.5h, where h is the total depth of the composite section.

f_{po} = $0.7f_{pu}$ for all sections taken away from beam ends a distance greater than the transfer length of the strands. For sections closer to the beam end, f_{po} shall vary linearly from zero at the beam end to $0.7f_{pu}$ at the end of the transfer length. However, shear resistance typically need not be calculated at sections within the transfer length.

E_s = Modulus of elasticity of non-prestressing steel (ksi)

A_s = Area of non-prestressing steel on flexural tension side of member (in.^{2}). For positive moment regions, A_s = 0 in.^{2}. For negative moment regions, A_s is taken as the area of the longitudinal reinforcement in the deck.

E_p = Modulus of elasticity of prestressing steel (ksi)

If ε_s is calculated as less than zero, it may be taken as zero, or more rigorous calculations may be employed.

Required Spacing of Transverse Reinforcement for Nominal Shear Resistance

As defined earlier, solving Equations 5.8.3.3-1, 5.8.3.3-3, C5.8.3.3-1, and 5.8.2.1-2 for s gives:

$$s \leq \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$

Where:

A_v = Area of two legs of transverse reinforcement (in.^{2}). Assume #4 bars are used.

f_y = Yield strength of transverse reinforcement (ksi)

d_v = Effective shear depth of section as defined above (in.)

θ = Angle of inclination of diagonal compressive stresses (degrees)

= $29 + 3500\varepsilon_s$, where ε_s is as defined above. (Eq. 5.8.3.4.2-3)

V_u = Factored Strength I shear at section under consideration (k)

ϕ = Resistance factor for shear as specified in Article 5.5.4.2

= 0.9

V_p = Vertical component of prestressing force as defined above (k)

V_c = Shear resistance due to concrete as defined above (k)

Maximum Permitted Spacing of Transverse Reinforcement *(5.8.2.7)*

Maximum spacing limits, s_{max} in in., are given by:

If $v_u < 0.125f'_c$, then:

$s_{max} = 0.8d_v \leq 24.0$ in. (Eq. 5.8.2.7-1)

If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4 d_v \leq 12.0 \text{ in.} \qquad \text{(Eq. 5.8.2.7-2)}$$

Where:

f'_c = Compressive strength of concrete (ksi)

v_u = Shear stress on concrete (ksi)

$$= \frac{\left|V_u - \phi V_p\right|}{\phi \, b_v d_v} \qquad \text{(Eq. 5.8.2.9-1)}$$

Where:

V_u = Factored Strength I shear at section under consideration (k)

V_p = Vertical component of prestressing force as defined above (k)

ϕ = Resistance factor for shear as specified in Article 5.5.4.2

= 0.9

b_v = Width of web of member (in.)

d_v = Effective shear depth of section as defined above (in.)

Minimum Transverse Reinforcement *(5.8.2.5)*

Solving Eq. 5.8.2.5-1 for s gives a maximum spacing of:

$$s = \frac{A_v f_y}{0.0316\sqrt{f'_c}\, b_v}$$

Where:

A_v = Area of two legs of transverse reinforcement (in.^{2}). Assume #4 bars are used.

f_y = Yield strength of transverse reinforcement (ksi)

f'_c = Compressive strength of concrete (ksi)

b_v = Width of web of member (in.)

While the specifications of Article 5.8.2.4 show that there are areas in a beam that do not require stirrups, since the standard stirrups are also used to make the beam and slab composite with one another, it is IDOT policy that at least the minimum amount of transverse reinforcement given by Article 5.8.2.5 be used in all areas of the beam.

Interface Shear Transfer Reinforcement *(5.8.4)*

The transverse reinforcement provided in prestressed beams to meet strength requirements is also used to meet interface shear reinforcement requirements by extending the transverse reinforcement across the interface between the top of the beam and the bottom of the slab.

When the required interface shear reinforcement in beam/slab design exceeds the area required to satisfy strength requirements, additional reinforcement shall be provided to satisfy interface shear requirements.

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \qquad \text{(Eq. 5.8.4.1-1)}$$

Where:

ϕ = Resistance factor for shear as specified in Article 5.5.4.2
= 0.9

V_{ni} = Nominal interface shear resistance as defined below (k)

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \qquad \text{(Eq. 5.8.4.1-3)}$$

But not greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv} \qquad \text{(Eq. 5.8.4.1-4)}$$

$$V_{ni} \leq K_2 A_{cv} \qquad \text{(Eq. 5.8.4.1-5)}$$

Where:

c = Cohesion factor specified in Article 5.8.4.3 (ksi), taken as 0.28 ksi for cast-in-place concrete slabs on clean concrete girder surfaces, free of laitance and roughened to an amplitude of 0.25 in. IDOT PPC I and bulb T-beams meet this criteria.

A_{cv} = Area of concrete considered to be engaged in interface shear transfer

= $b_{vi}L_{vi}$, (Eq. 5.8.4.1-6)

Where:

b_{vi} = Beam top flange width (in.)

L_{vi} = 12 inch length along beam

μ = Friction factor specified in Article 5.8.4.3, taken as 1.0 for IDOT PPC I and bulb T-beams.

A_{vf} = Area of two legs of transverse reinforcement (in.^{2})

f_y = Yield strength of transverse reinforcement (ksi)

P_c = Permanent net compressive force normal to the shear plane, taken as zero (k)

K_1 = Fraction of concrete strength available to resist interface shear as specified in Article 5.8.4.3, taken as 0.3 for IDOT PPC I and bulb T-beams.

K_2 = Limiting interface shear resistance specified in Article 5.8.4.3 (ksi), taken as 1.8 ksi for IDOT PPC I and bulb T-beams.

Substituting $\frac{12A_{vf}}{s}$ for A_{vf} and solving Equations 5.8.4.1-1 and 5.8.4.1-3 for s and setting V_{ri} equal to V_{ui} gives the following equation for maximum spacing:

$$s = \frac{12\mu A_{vf} f_y}{\frac{V_{ui}}{\phi} - cA_{cv}} \quad \text{(Eq. i)}$$

Where:

V_{ui} = $v_{ui}A_{cv}$ (Eq. 5.8.4.2-2)

Where:

$$v_{ui} = \frac{V_u}{b_{vi}d_v} \quad \text{(Eq. 5.8.4.2-1)}$$

Where V_u is the factored Strength I shear in the vertical direction, and b_{vi} and d_v are as defined above.

When using this equation for s, the following modifications of Equations 5.8.4.1-4 and 5.8.4.1-5 are required to be checked:

$$\frac{V_{ui}}{\phi} \leq K_1 f'_c A_{cv}$$

$$\frac{V_{ui}}{\phi} \leq K_2 A_{cv}$$

If either of these two requirements is not satisfied, the lesser value of $K_1 f'_c A_{cv}$ or $K_2 A_{cv}$ should be substituted for $\frac{V_{ui}}{\phi}$ when calculating the required spacing for interface shear.

Per Article 5.8.4.4, the minimum required area of interface shear reinforcement need not exceed the lesser of:

$$A_{vf} = \frac{0.05A_{cv}}{f_y} \qquad \text{(Eq. 5.8.4.4-1)}$$

and

the amount needed to resist $\frac{1.33V_{ui}}{\phi}$ as determined using Eq. 5.8.4.1-3.

Equation 5.8.4.4-1 can be modified by substituting $\frac{12A_{vf}}{s}$ for A_{vf} and solving for s, giving the following equation for maximum spacing:

$$s = \frac{12A_{vf} f_y}{0.05A_{cv}} \qquad \text{(Eq. ii)}$$

The spacing needed to resist $\frac{1.33V_{ui}}{\phi}$ can be determined by substituting $\frac{1.33V_{ui}}{\phi}$ for $\frac{V_{ui}}{\phi}$ into equation i, giving the following equation for maximum spacing:

$$s = \frac{12\mu A_{vf} f_y}{\frac{1.33V_{ui}}{\phi} - cA_{cv}} \qquad \text{(Eq. iii)}$$

Since Equations ii and iii are based on determination of minimum reinforcement, the greater spacing given by these two equations will control the check.

In many situations no reinforcement is required for interface shear.

Longitudinal Reinforcement *(5.8.3.5)*

LRFD Article 5.8.3.5 shall also be checked. The requirements state that the tensile capacity of the longitudinal reinforcement should be greater than the calculated tension based upon the design shear and moment.

The LRFD code states that this check should be made near supports. Outlined below are methods to employ near simple supports or abutments and near continuous supports or piers.

Abutments:

At sections near abutments, longitudinal reinforcement requirements should be checked at the inside edge of bearing and at the critical section. For beams at fixed abutments, the edge of bearing may be taken as the face of the support.

The requirement for longitudinal reinforcement is given by:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad \text{(Eq. 5.8.3.5-2)}$$

Where:

A_s = Area of non-prestressed tension reinforcement on flexural tension side of member (in.^{2}), taken as zero for abutments.

f_y = Yield strength of non-prestressed reinforcement (ksi)

A_{ps} = Area of prestressing steel on flexural tension side of member (in.^{2}) as defined above.

f_{ps} = Average stress in prestressing steel (ksi) as defined above.

$|V_u|$ = Factored Strength I shear (k). For sections at faces of abutments, $|V_u|$ may conservatively be taken as $|V_u|$ at the support.

ϕ_v = Resistance factor for shear as specified in Article 5.5.4.2

= 0.9

V_s = Shear resistance due to transverse reinforcement (k), not to be taken as larger than $\frac{|V_u|}{\phi_v}$

V_p = Vertical component of prestressing force as defined above (k)

θ = Angle of inclination of diagonal compressive stresses (degrees) as defined above

As V_u is higher at CL bearing locations than at critical sections, and V_p is higher at critical sections, it is conservative and simpler to perform this check only once, using V_u at the CL bearing location and V_p at the critical section. This will also avoid recalculation of d_v for beams with harped strands.

<u>Piers:</u>

At sections near piers, the longitudinal reinforcement check need not be performed if the flexural reinforcement in the slab is extended a distance of $d_v \cot \Theta$ beyond the critical section and the requirements of 5.11.1.2.3 are satisfied. As the requirements of 5.11.1.2.3 are satisfied during the negative moment design, and it is IDOT policy for additional flexural reinforcement at piers to extend at least a distance of 0.2 times the span length into the span, checking the following equation is sufficient in lieu of performing full calculations:

$$\frac{d_v(1+\cot\theta)+x_{face}}{L} < 0.2 \qquad \text{(C5.8.3.5, modified)}$$

Where d_v and θ are as calculated above, L is the span length (in.), and x_{face} is the distance from the centerline of the pier to the face of the support (in.).

If this is not satisfied, the requirement for longitudinal steel shall be checked at the face of the pier and at the critical section, and is given by:

$$A_s f_y + A_{ps} f_{ps} \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta \qquad \text{(Eq. 5.8.3.5-1)}$$

Where:

A_s = Area of non-prestressed tension reinforcement on flexural tension side of member (in.^{2}), taken as the area of longitudinal slab reinforcement for negative moment areas.

f_y = Yield strength of non-prestressed reinforcement (ksi)

A_{ps} = Area of prestressing steel on flexural tension side of member (in.^{2}) as defined above.

f_{ps} = Average stress in prestressing steel (ksi) as defined above.

$|M_u|$ = Absolute factored Strength I moment (k-in.). For sections at faces of piers, $|M_u|$ may conservatively be taken at the centerline of pier.

d_v = Effective shear depth, as defined above (in.)

ϕ_f = Resistance factor for flexure as specified in Article 5.5.4.2 for non-prestressed reinforcement.

For $\varepsilon_t \geq 0.005$, the full value of $\phi = 0.9$ is used. (Fig. C5.5.4.2.1-1)

For $0.002 \geq \varepsilon_t > 0.005$, $\phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right)$ (Eq. 5.5.4.2.1-2)

For $\varepsilon_t < 0.002$, $\phi = 0.75$ (Fig. C5.5.4.2.1-1)

Where:

ε_t = Tensile strain in extreme reinforcing element at point when concrete strain equals 0.003 in./in. (in./in.)

d_t = Distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.).

c = Distance from extreme compression fiber to neutral axis (in.). See moment design for computation of c.

As harped prestressing strands in the tension side of negative moment regions are not used in strength calculations, using the Partial Prestressing Ratio to increase ϕ_f is not allowed.

$|V_u|$ = Factored Strength I shear (k)

ϕ_v = Resistance factor for shear as specified in Article 5.5.4.2

= 0.9, assuming non-prestressed concrete

V_s = Shear resistance due to transverse reinforcement (k), not to be taken as larger than $\frac{|V_u|}{\phi_v}$

V_p = Vertical component of prestressing force as defined above (k)

θ = Angle of inclination of diagonal compressive stresses (degrees) as defined above.

Check Final Design Against Eq. 5.8.3.3-2

$$V_n = 0.25f'_c b_v d_v + V_p \qquad \text{(Eq. 5.8.3.3-2)}$$

Where:

f'_c = Compressive strength of concrete (ksi)

b_v = Width of web of member (in.)

d_v = Effective shear depth of section as defined above (in.)

V_p = Vertical component of prestressing force as defined above (k)

Example 1

90 ft., single span, 54 inch PPC I-beam, 7 beam lines, 6.25 ft. beam spacing, 8 in. deck, F-shape barrier, 50 pounds per square foot future wearing surface, no skew, 3 design lanes and HL-93 loading on integral abutments. It should be noted that the non-composite span lengths were conservatively set equal to the composite span lengths for simplicity.

General Data

Design Code	=	LRFD (simplified live load distribution)
Shear Design Method	=	General Procedure (Article 5.8.3.4.2)
Span length	=	90 ft.
Beam section	=	54 in. PPC I-beam
Beam spacing	=	6.25 ft.
Number of beams	=	7
Deck thickness	=	8 in.
Estimated camber	=	2 in.
Average fillet	=	1.5 in. (based on ½ in. min. fillet and estimated camber)
Parapet	=	0.45 k/ft.
FWS	=	50 psf
Relative Humidity	=	70
Strands	=	½ in. diameter – 270 ksi low relaxation strands
Skew	=	0 degrees

Live Load Data

Loading	=	HL-93
IM	=	1.33 (HL-93); 1.15 (fatigue truck)

N_L = 3

Trial Strand Pattern

Select strand pattern 28DS from planning charts in BM Section 2.3.6.1.3.

$$A_{ps} = 28\left(0.153 \text{ in.}^2\right) = 4.284 \text{ in.}^2$$

Materials

Precast Concrete Beam

f'_c = 6.0 ksi

f'_{ci} = 5.0 ksi

f_{pbt} = 201.96 ksi

f_{pu} = 270.0 ksi

Fi/strand = 30.9 kips

Cast in Place Concrete Deck

f'_c = 3.5 ksi

f_y = 60.0 ksi

Section Properties

Modulus of Elasticity

$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c}$ (Eq. 5.4.2.4-1)

$E_{ci} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ci}}$ (Eq. 5.4.2.4-1)

$E_c \text{ (deck)} = 33{,}000(1.0)(0.15 \text{ k/ft.})^{1.5}\sqrt{3.5 \text{ ksi}} = 3587 \text{ ksi}$

$E_{ci} \text{ (beam)} = 33{,}000(1.0)(0.15 \text{ k/ft.})^{1.5}\sqrt{5.0 \text{ ksi}} = 4287 \text{ ksi}$

$E_c \text{ (beam)} = 33{,}000(1.0)(0.15 \text{ k/ft.})^{1.5}\sqrt{6.0 \text{ ksi}} = 4696 \text{ ksi}$

E_p (strand) = 28500 ksi (5.4.4.2)

Non-Composite (Beam only)

A = 599 in.^2

I = 213715 in.^4

S_b = 8559 in.^3 C_b = 24.97 in

S_t = 7362 in.^3 C_t = 29.03 in.

Composite

Modular Ratio:

$$n = \frac{E_c(\text{deck})}{E_c(\text{beam})} = \frac{3587\text{ ksi}}{4696\text{ ksi}} = 0.76$$

Effective Flange Width: (4.6.2.6)

Dist. C. to C. of beams = 6.25 ft.(12 in./ft.) = 75 in.

Compute composite section properties:

		A	y	Ay
Slab	0.76(75 in.)(8 in.)	456.0 in.^{2}	58.00 in.	26448 in.^{3}
Beam		599.0 in.^{2}	24.97 in.	14957 in.^{3}
		1055.0 in.^{2}		41405 in.^{3}

$$y_b = \frac{41405\text{ in.}^3}{1055\text{ in.}^2} = 39.25\text{ in.}$$

$$y_t = 54\text{ in.} - 39.25\text{ in.} = 14.75\text{ in.}$$

	Io	A	d	Ad^2	I'
Slab	2432 in.^{4}	456.0 in.^{2}	18.75 in.	160313 in.^{4}	162745 in.^{4}
Beam	213715 in.^{4}	599.0 in.^{2}	14.28 in.	122147 in.^{4}	335862 in.^{4}
				I' =	498607 in.^{4}

$$S'_b = \frac{498607\text{ in.}^4}{39.25\text{ in.}} = 12703\text{ in.}^3$$

$$S'_t = \frac{498607\text{ in.}^4}{14.75\text{ in.}} = 33804\text{ in.}^3$$

C = 1.10

Moment

$$g_1 = 0.06 + C\left(\frac{S}{14.0}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3} = 0.06 + 1.10\left(\frac{6.25\text{ ft.}}{14.0}\right)^{0.4}\left(\frac{6.25\text{ ft.}}{90\text{ ft.}}\right)^{0.3}$$

$$= 0.418$$

$$g_m = 0.075 + C\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2} = 0.075 + 1.10\left(\frac{6.25\text{ ft.}}{9.5}\right)^{0.6}\left(\frac{6.25\text{ ft.}}{90\text{ ft.}}\right)^{0.2}$$

$= 0.577$ <= Controls

Moment (fatigue loading)

$$g_1\text{ (fatigue)} = \frac{g_1}{m} = \frac{0.418}{1.2} = 0.348$$

Shear and Reaction

$$g_1 = 0.36 + \frac{S}{25.0} = 0.36 + \frac{6.25\text{ ft.}}{25.0}$$

$$= 0.610$$

$$g_m = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} = 0.2 + \left(\frac{6.25\text{ ft.}}{12}\right) - \left(\frac{6.25\text{ ft.}}{35}\right)^{2.0}$$

$= 0.689$ <= Controls

Skew correction $= 1 + 0.2\tan(\theta)$

$= 1 + 0.2\tan(0)$

$= 1.000$

Deflection

$$g\ (\text{deflection}) = m\left(\frac{N_L}{N_b}\right) = 0.85\left(\frac{3}{7}\right) = .364$$

Dead Loads

Non-Composite

DC1:

Beam $= 0.624$ k/ft.

Slab $\left(0.15\ \text{k/ft.}^3\right)\left(\frac{8\ \text{in}}{12\ \text{in./ft.}}\right)\left(6.25\ \text{ft.}\right) = 0.625$ k/ft.

Fillet $\left(0.15\ \text{k/ft.}^3\right)\left(\frac{1.5\ \text{in.}}{12\ \text{in./ft.}}\right)\left(\frac{20\ \text{in.}}{12\ \text{in./ft.}}\right) = 0.031$ k/ft.

DW1:

No non-composite dead loads in this category.

Composite

DC2:

Parapets $\frac{\left(0.45\ \text{k/ft.}\right)\left(2\ \text{parapets}\right)}{7\ \text{beams}} = 0.129$ k/ft.

DW2:

FWS $\left(0.050\ \text{k/ft.}^2\right)\left(6.25\ \text{ft.}\right) = 0.313$ k/ft.

Maximum Unfactored Distributed Moments

All moments generated by computer software.

Non-Composite

DC1:

@0.5L $M_{DC1} = 1296$ k-ft.

DW1:

No non-composite DW1 loads.

Composite

DC2:

@0.5L M_{DC2} = 131 k-ft.

DW2:

@0.5L M_{DW2} = 317 k-ft.

LL+IM:

@0.5L M_{LL+IM} = 1402 k-ft.

@0.5L M_{FL+IM} = 434 k-ft

Lifting Loop Design

Beam length = 91.0 ft.

Total beam weight= $(0.624\ \text{k/ft.})(91\ \text{ft.})$ = 57.0 kips

From the charts: 2 - 3 strand lifting loops required each end at 3.25 ft. and 6.25 ft.

Strand Eccentricities

The length of beam is used instead of the length of span when calculating the eccentricity along draped strands.

Eccentricity at center:

Row	Number (N)	m	N(m)
1	10	2 in.	20 in.
2	10	4 in.	40 in.
3	8	6 in.	48 in.
Total	28		108 in.

C.G. of strands measured from bottom = $\frac{108\ \text{in.}}{28}$ = 3.86 in.

e = $C_b - 3.86\ \text{in.}$ = $24.97\ \text{in.} - 3.86\ \text{in.}$ = 21.11 in.

Eccentricity at end:

Row	Number (N)	m	N(m)
1	8	2 in.	16 in.
2	8	4 in.	32 in.
3	6	6 in.	36 in.
3T	2	47 in.	94 in.
2T	2	49 in.	98 in.
1T	2	51 in.	102 in.
Total	28		378 in.

C.G. of strands measured from bottom $= \dfrac{378 \text{ in.}}{28} = 13.50$ in.

$e = C_b - 13.50 \text{ in.} = 24.97 \text{ in.} - 13.50 \text{ in.} = 11.47$ in.

Eccentricity at center of lifting loops (use average of 3.25 ft. and 6.25 ft. = 4.75 ft.):

Row	Number (N)	m	N(m)
1	8	2 in.	16 in.
2	8	4 in.	32 in.
3	6	6 in.	36 in.
3T	2	41.1 in.	82.2 in.
2T	2	43.1 in.	86.2 in.
1T	2	45.1 in.	90.2 in.
Total	28		342.6 in.

C.G. of strands measured from bottom $= \dfrac{342.6 \text{ in.}}{28} = 12.24$ in.

$e = C_b - 12.24 \text{ in.} = 24.97 \text{ in.} - 12.24 \text{ in.} = 12.73$ in.

Prestress Losses

Total Loss of Prestress

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.5.1-1)

Instantaneous Losses (due to elastic shortening):

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ (Eq. 5.9.5.2.3a-1)

Assume F_t equals 90 percent of F_i for first iteration:

$$F_i = A_{ps}(f_{pbt}) = 4.284 \text{ in.}^2 (201.96 \text{ ksi}) = 865 \text{ kips}$$

$$F_t = 0.9(F_i) = 0.9(865 \text{ kips}) = 779 \text{ kips}$$

$$M_b \text{ (beam)} = \frac{(0.624 \text{ k/ft.})(90 \text{ ft.})^2}{8} = 632 \text{ k-ft.}$$

$$f_{cgp} = \frac{F_t}{A} + \frac{F_t e^2}{I} - \frac{M_b (12) e}{I}$$

$$= \frac{779 \text{ kips}}{599 \text{ in.}^2} + \frac{(779 \text{ kips})(21.11 \text{ in.})^2}{213715 \text{ in.}^4} - \frac{(632 \text{ k-ft.})(12 \text{ in./ft.})(21.11 \text{ in.})}{213715 \text{ in.}^4}$$

$$= 2.18 \text{ ksi}$$

Calculate Δf_{pES}:

$$\Delta f_{pES} = \frac{28500 \text{ ksi}}{4287 \text{ ksi}} (2.18 \text{ ksi}) = 14.49 \text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} = \frac{(201.96 \text{ ksi} - 14.49 \text{ ksi})}{201.96 \text{ ksi}} = 0.93 > 0.90$$

Recalculate

Assume F_t equals 93 percent of F_i for second iteration:

$$F_t = 0.93(865 \text{ kips}) = 805 \text{ kips}$$

$$f_{cgp} = \frac{805\text{ kips}}{599\text{ in.}^2} + \frac{(805\text{ kips})(21.11\text{ in.})^2}{213715\text{ in.}^4} - \frac{(632\text{ k}-\text{ft.})(12\text{ in./ft.})(21.11\text{ in.})}{213715\text{ in.}^4}$$

$$= 2.27\text{ ksi}$$

Calculate Δf_{pES}:

$$\Delta f_{pES} = \frac{28500\text{ ksi}}{4287\text{ ksi}}(2.27\text{ ksi}) = 15.09\text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} = \frac{(201.96\text{ ksi} - 15.09\text{ ksi})}{201.96\text{ ksi}} = 0.93 \quad \text{Ok}$$

Time Dependent Losses:

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(Eq. 5.9.5.3-1)}$$

In which:

$$\gamma_h = 1.7 - 0.01H = 1.7 - 0.01(70) = 1.0 \qquad \text{(Eq. 5.9.5.3-2)}$$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} = \frac{5}{(1+5\text{ ksi})} = 0.833 \qquad \text{(Eq. 5.9.5.3-3)}$$

$$\Delta f_{pR} = 2.4\text{ ksi}$$

$$\Delta f_{pLT} = 10.0\frac{(201.96\text{ ksi})(4.284\text{ in.}^2)}{599\text{ in.}^2}(1.0)(0.833) + 12.0(1.0)(0.833) + 2.4\text{ ksi} = 24.43\text{ ksi}$$

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 15.09\text{ ksi} + 24.43\text{ ksi} = 39.52\text{ ksi}$$

$$\%\text{Loss} = \frac{39.52\text{ ksi}}{201.96\text{ ksi}} = 19.6\ \%$$

Stress Limits for Concrete

Temporary stresses (5.9.4.1)

Compression:

$$0.60f'_{ci} = 0.60(5.0\text{ ksi}) = 3.00\text{ ksi}$$

Tension:

$$0.24\sqrt{f'_{ci}} = 0.24\sqrt{5.0\text{ ksi}} = 0.537\text{ ksi}$$

Service stresses after losses (5.9.4.2)

Compression (For Service I load combination):

$$0.60\phi_w f'_c = 0.60(1.0)(6.0\text{ ksi}) = 3.60\text{ ksi} \quad \text{(a)}$$

$$0.45f'_c = 0.45(6.0\text{ ksi}) = 2.70\text{ ksi} \quad \text{(b)}$$

Tension (For Service III load combination):

$$0.19\sqrt{f'_c} = 0.19\sqrt{6.0\text{ ksi}} = 0.465\text{ ksi}$$

Fatigue stresses after losses (5.5.3.1)

Compression (For Fatigue I load combination):

$$0.40f'_c = 0.40(6.0\text{ ksi}) = 2.40\text{ ksi}$$

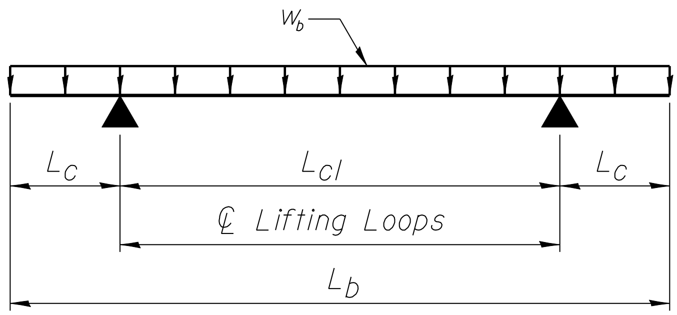
Tension limit for determination of cracked verse uncracked section:

$$\text{Uncracked} \leq 0.095\sqrt{f'_c} \leq \text{Cracked}$$

$$0.095\sqrt{6.0\text{ ksi}} = 0.233\text{ ksi}$$

Check Temporary Stresses

Calculate Dead Load Moments for Determining Temporary Stresses



@ Lifting Loops:

$$M_{bts} = \frac{w_b(L_c)^2}{2} = \frac{(0.624\text{ k/ft.})(4.75\text{ ft.})^2}{2} = 7\text{ k-ft.}$$

@ Harping Point (0.4 L_b):

$$M_{bts} = \frac{3w_bL_b^2}{25} - \frac{w_bL_bL_c}{2}$$

$$= \frac{3(0.624\text{ k/ft.})(91.0\text{ ft.})^2}{25} - \frac{(0.624\text{ k/ft.})(91.0\text{ ft.})(4.75\text{ ft.})}{2} = 485\text{ k-ft.}$$

Prestress Force Immediately after Transfer

$$F_t = A_{ps}(f_{pbt} - \Delta f_{pES}) = (4.284\text{ in.}^2)(201.96\text{ ksi} - 15.09\text{ ksi}) = 801\text{ kips}$$

Temporary Stresses

@ Lifting Loops:

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} - \frac{M_{bts}(12)}{S_t}$$

$$= \frac{801\text{ kips}}{599\text{ in.}^2} - \frac{(801\text{ kips})(12.73\text{ in.})}{7362\text{ in.}^3} - \frac{(7\text{ k} - \text{ft.})(12\text{ in./ft.})}{7362\text{ in.}^3}$$

$= 0.059$ ksi (tension) ≤ 0.537 ksi Ok

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} + \frac{M_{bts}(12)}{S_b}$$

$$= \frac{801\,\text{kips}}{599\,\text{in.}^2} + \frac{(801\,\text{kips})(12.73\,\text{in.})}{8559\,\text{in.}^3} + \frac{(7\,\text{k}-\text{ft.})(12\,\text{in./ft.})}{8559\,\text{in.}^3}$$

= 2.538 ksi (comp.) ≤ 3.000 ksi Ok

@ Harping Point (0.4 L_b):

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} + \frac{M_{bts}(12)}{S_t}$$

$$= \frac{801\,\text{kips}}{599\,\text{in.}^2} - \frac{(801\,\text{kips})(21.11\,\text{in.})}{7362\,\text{in.}^3} + \frac{(485\,\text{k}-\text{ft.})(12\,\text{in./ft.})}{7362\,\text{in.}^3}$$

= 0.169 ksi (tension) ≤ 0.537 ksi Ok

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} - \frac{M_{bts}(12)}{S_b}$$

$$= \frac{801\,\text{kips}}{599\,\text{in.}^2} + \frac{(801\,\text{kips})(21.11\,\text{in.})}{8559\,\text{in.}^3} - \frac{(485\,\text{k}-\text{ft.})(12\,\text{in./ft.})}{8559\,\text{in.}^3}$$

= 2.633 ksi (comp.) ≤ 3.000 ksi Ok

Design Positive Moment Region

<u>Check Service Stresses after Losses</u>

Prestress Force after Losses:

$$F_s = A_{ps}(f_{pbt} - \Delta f_{pT}) = (4.284\,\text{in.}^2)(201.96\,\text{ksi} - 39.52\,\text{ksi}) = 696\,\text{kips}$$

Service Stresses:

@ Center:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t'} \quad \text{(a)}$$

$$= \frac{696\,\text{kips}}{599\,\text{in.}^2} - \frac{(696\,\text{kips})(21.11\,\text{in.})}{7362\,\text{in.}^3} + \frac{(1296\,\text{k}-\text{ft.}+0)(12\,\text{in./ft.})}{7362\,\text{in.}^3}$$

$$+ \frac{(131\,\text{k}-\text{ft.} + 317\,\text{k}-\text{ft.} + 1402\,\text{k}-\text{ft.})(12\,\text{in./ft.})}{33804\,\text{in.}^3}$$

= 1.935 ksi (comp.) ≤ 3.600 ksi Ok

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \qquad \text{(b)}$$

$$= \frac{696 \text{ kips}}{599 \text{ in.}^2} - \frac{(696 \text{ kips})(21.11 \text{ in.})}{7362 \text{ in.}^3} + \frac{(1296 \text{ k}-\text{ft.} + 0)(12 \text{ in./ft.})}{7362 \text{ in.}^3}$$

$$+ \frac{(131 \text{ k}-\text{ft.} + 317 \text{ k}-\text{ft.})(12 \text{ in./ft.})}{33804 \text{ in.}^3}$$

= 1.438 ksi (comp.) ≤ 2.700 ksi Ok

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

$$= \frac{696 \text{ kips}}{599 \text{ in.}^2} + \frac{(696 \text{ kips})(21.11 \text{ in.})}{8559 \text{ in.}^3} - \frac{(1296 \text{ k}-\text{ft.} + 0)(12 \text{ in./ft.})}{8559 \text{ in.}^3}$$

$$- \frac{(131 \text{ k}-\text{ft.} + 317 \text{ k}-\text{ft.})(12 \text{ in./ft.})}{12703 \text{ in.}^3} - (0.8)\frac{(1402 \text{ k}-\text{ft.})(12 \text{ in./ft.})}{12703 \text{ in.}^3}$$

= 0.421 ksi (tension) ≤ 0.465 ksi Ok

Check Fatigue Stresses after Losses

Determine if section is cracked for fatigue investigions:

@ Center:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (1.5)\frac{M_{FL+IM}(12)}{S_b^{'}}$$

$$= \frac{696 \text{ kips}}{599 \text{ in.}^2} + \frac{(696 \text{ kips})(21.11 \text{ in.})}{8559 \text{ in.}^3} - \frac{(1296 \text{ k}-\text{ft.} + 0)(12 \text{ in./ft.})}{8559 \text{ in.}^3}$$

$$- \frac{(131 \text{ k}-\text{ft.} + 317 \text{ k}-\text{ft.})(12 \text{ in./ft.})}{12703 \text{ in.}^3} - (1.5)\frac{(434 \text{ k}-\text{ft.})(12 \text{ in./ft.})}{12703 \text{ in.}^3}$$

= 0.023 ksi (comp.) ≤ 0.233 ksi Use uncracked section properties

Fatigue Stresses:

@ Center:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_t^{'}}$$

$$= 0.5\left(\frac{696\text{ kips}}{599\text{ in.}^2}\right) - 0.5\frac{(696\text{ kips})(21.11\text{ in.})}{7362\text{ in.}^3} + 0.5\frac{(1296\text{ k-ft.} + 0)(12\text{ in./ft.})}{7362\text{ in.}^3}$$

$$+ 0.5\frac{(131\text{ k-ft.} + 317\text{ k-ft.})(12\text{ in./ft.})}{33804\text{ in.}^3} + 1.5\frac{(434\text{ k-ft.})(12\text{ in./ft.})}{33804\text{ in.}^3}$$

= 0.950 ksi (comp.) ≤ 2.400 ksi Ok

<u>Check Factored Flexural Resistance</u>

Strength I Moment:

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

= 1.25(1296 k-ft. + 131 k-ft.) + 1.5(0 k-ft. + 317k-ft.) + 1.75(1402 k-ft.)

= 4713 k-ft.

Factored Flexural Resistance:

$M_r = \phi M_n$

Calculate Compression Block Depth (assume rectangular):

$$c = \frac{A_{ps}f_{pu}}{0.85f'_c\beta_1 b + kA_{ps}\frac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.7.3.1.1-4)}$$

In which:

f'_c = deck concrete strength = 3.5 ksi

$\beta_1 = 0.65 \le 0.85 - 0.05(f'_c - 4.0) \le 0.85$ (5.7.2.2)

$= 0.65 \le 0.85 - 0.05(3.5\text{ ksi} - 4.0) \le 0.85$

$= 0.65 \le 0.875 \le 0.85$

= 0.85

b = effective flange width = 75 in.

k = 0.28

d_p = deck thickness + C_t + e (center)

= 8 in.+ 29.03in. + 21.11in.

= 58.14 in.

$$c = \frac{(4.284 \text{ in.}^2)(270 \text{ ksi})}{0.85(3.5 \text{ ksi})(0.85)(75 \text{ in.}) + 0.28(4.284 \text{ in.}^2)\frac{270 \text{ ksi}}{58.14 \text{ in.}}}$$

$= 5.92$ in.

$a = \beta_1 c$

$= 0.85(5.92 \text{ in.})$

$= 5.03 \text{ in.} \leq 8 \text{ in.}$ Therefore Rectangular Section

Calculate Nominal Flexural Resistance:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)\left(\frac{1}{12}\right) \qquad \text{(Eq. 5.7.3.2.2-1)}$$

In which:

Check $f_{pe} \geq 0.5 f_{pu}$ to verify use of Eq. 5.7.3.1.1-1 (5.7.3.1.1)

$f_{pe} = f_{pu} - \Delta f_{pT}$

$= 270 \text{ ksi} - 39.52 \text{ ksi}$

$= 230.48 \text{ ksi} \geq 0.5(270 \text{ ksi}) = 135 \text{ ksi}$ Ok

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(Eq. 5.7.3.1.1-1)}$$

$$= 270 \text{ ksi}\left(1 - 0.28\frac{5.92 \text{ in.}}{58.14 \text{ in.}}\right)$$

$= 262$ ksi

$$M_n = \left[(4.284 \text{ in.}^2)(262 \text{ ksi})\left(58.14 \text{ in.} - \frac{5.03 \text{ in.}}{2}\right)\right]\left(\frac{1}{12 \text{ in./ft.}}\right)$$

$= 5203$ k-ft.

Calculate ϕ:

$$\phi = 0.75 \leq 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \leq 1.0 \qquad \text{(Eq. 5.5.4.2.1-1)}$$

In which:

d_t = deck thickness + depth of beam − distance from bottom of beam to bottom row of strands

= 8 in. + 54 in. − 2 in.

= 60 in.

$$\phi = 0.75 \le 0.583 + 0.25\left(\frac{60\text{ in.}}{5.92\text{ in.}} - 1\right) \le 1.0$$

$$= 0.75 \le 2.87 \le 1.0$$

$$= 1.0$$

M_r = 1.0(5203 k-ft.)

= 5203 k-ft. ≥ 4713 k-ft. Ok

Check Minimum Prestressing Steel:

$M_r \geq M_{cr}$ (5.7.3.3.2)

In which:

$$f_r = 0.24\sqrt{f'_c} \quad (5.4.2.6)$$

$$= 0.24\sqrt{6.0\text{ ksi}}$$

= 0.59 ksi

$$f_{cpe} = \frac{F_s}{A} + \frac{F_s e}{S_b}$$

$$= \frac{696\text{ kips}}{599\text{ in.}^2} + \frac{(696\text{ kips})(21.11\text{ in.})}{8559\text{ in.}^3}$$

= 2.88 ksi

$$M_{cr} = \gamma_3\left[\frac{S'_b(\gamma_1 f_r + \gamma_2 f_{cpe})}{12} - M_{DC1}\left(\frac{S'_b}{S_b} - 1\right)\right] \quad \text{(Eq. 5.7.3.3.2-1)}$$

$$= 1.00\left[\frac{(12703\text{ in.}^3)(1.6(0.59\text{ ksi}) + 1.1(2.88\text{ ksi}))}{(12\text{ in./ft.})} - 1296\text{ k-ft.}\left(\frac{12703\text{ in.}^3}{8559\text{ in.}^3} - 1\right)\right]$$

= 3725 k-ft.

5203 k-ft. ≥ 3725 k-ft. Ok

Calculate Camber and Deflection

Initial Resultant Camber

Camber = D_{cp} - D_{cb}

In which:

$$D_{cp} = \frac{F_t(12L)^2}{E_{ci}I}\left[0.0983\,e_{center} + 0.0267\,e_{end}\right](1.80) \quad \text{for draped strand patterns}$$

$$= \frac{(801\text{ kips})\left[(12\text{ in./ft.})(90\text{ ft.})\right]^2}{(4287\text{ ksi})(213715\text{ in.}^4)}\left[0.0983(21.11\text{ in.}) + 0.0267(11.47\text{ in.})\right](1.80)$$

= 4.37 in. up

$$D_{cb} = \frac{5\,w_b(12L)^4}{384(12)\,E_{ci}\,I}(1.85)$$

$$= \frac{5(0.624\text{ k/ft.})\left[(12\text{ in./ft.})(90\text{ ft.})\right]^4}{384(12\text{ in./ft.})(4287\text{ ksi})(213715\text{ in.}^4)}(1.85)$$

= 1.86 in. down

Camber = 4.37 in.-1.86 in.

= 2.51 in. up

Final Resultant Camber for Computing Bearing Seat Elevations

Camber = D_{cp} - D_{cb} - D_{cs}

In which:

$$D_{cs} = \frac{5\,w_s(12L)^4}{384(12)\,E_c\,I}$$

$$= \frac{5(0.656\text{ k/ft.})\left[(12\text{ in./ft.})(90\text{ ft})\right]^4}{384(12\text{ in./ft.})(4696\text{ ksi})(213715\text{ in.}^4)}$$

= 0.96 in. down

Camber = 4.37 in.-1.86in.-0.96 in.

= 1.55 in. up

Downward Deflections Due to Slab Weight for Adjusting Grade Elevations

@0.25 point = $0.7125D_{cs}$ = 0.7125(0.96 in.) = 0.68 in.

@0.50 point = D_{cs} = 0.96 in.

@0.75 point = $0.7125D_{cs}$ = 0.7125(0.96 in.) = 0.68 in.

Shear Design *(5.8)*

As showing full calculations in this design guide for every location along the beam is lengthy and unnecessary, full shear calculations will be shown for the critical section, and only tabulated results will be shown for the rest of the sections along the beam.

Location of Critical Section *(5.8.3.2)*

Taking the location of the critical section for shear at 0.72h from the face of the support gives a location of:

x_{Crit} = 0.72h + 1.25 ft. (face of support to CL bearing)

$$= 0.72(54\text{ in. beam} + 8\text{ in. slab})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) + 1.25\text{ ft.}$$

= 4.97 ft. along span

The calculated values of the maximum and minimum Strength I shears and moments, factored, distributed, and including impact, at this location have been found to be:

M_u^+ = 1012.1 k-ft. (from computer software)

M_u^- = 310.9 k-ft. (from computer software)

V_u^+ = 217.4 k-ft. (from computer software)

V_u^- = 56 k-ft. (from computer software)

Nominal Shear Resistance *(5.8.3.3)*

The maximum permitted spacing based upon nominal shear resistance is taken as:

$$s = \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$

Where the required variables are as calculated below.

Effective Shear Depth d_v *(5.8.2.9)*

The effective shear depth, d_v, is taken as:

$$d_v = d_e - \frac{a}{2} \qquad \text{(C5.8.2.9)}$$

Where:

d_e = $h - (C_b - e_{Crit})$

Where:

h = 54 in. beam + 8 in. slab = 62 in.

C_b = 24.97 in. (see Bridge Manual Table 3.4.4.2-1)

e_{Crit} = Eccentricity of strand group at critical section

$$= e_{End} + \left(\frac{x_{Crit} + x_{brg}}{0.4L_b}\right)(e_{Center} - e_{End})$$

Where:

e_{End} = 11.470 in. (see Bridge Manual Table 3.4.4.1-7)

x_{Crit} = 4.97 ft.

x_{brg} = distance from end of beam to CL bearing (ft.)

= 0.5 ft.

L_b = 91 ft.

e_{Center} = 21.113 in. (see Bridge Manual Table 3.4.4.1-7)

$$e_{Crit} = 11.470 \text{ in.} + \left(\frac{4.97 \text{ ft.} + 0.5 \text{ ft.}}{0.4(91 \text{ ft.})}\right)(21.113 \text{ in.} - 11.470 \text{ in.})$$

$$= 12.919 \text{ in.}$$

$$d_e = 62 \text{ in.} - (24.97 \text{ in.} - 12.919 \text{ in.})$$

$$= 49.95 \text{ in.}$$

In moment calculations, "a" was found to be 5.03 in. at x = 0.5L. Since the position of the harped strands does not affect the depth of the stress block in positive moment regions, this number is also valid at the critical section.

$$d_v = 49.95 \text{ in.} - \left(\frac{5.03 \text{ in.}}{2}\right) \qquad \text{(C5.8.2.9)}$$

$$= 47.44 \text{ in.}$$

$$0.9d_e = 0.9(49.95 \text{ in.})$$

$$= 44.95 \text{ in.}$$

$$0.72h = 0.72(62 \text{ in.})$$

$$= 44.64 \text{ in.}$$

Since d_v is greater than both $0.9d_e$ and 0.72h, it is the controlling value.

$$d_v = 47.44 \text{ in.}$$

Vertical Component of Prestressing Force V_p

$$V_p = A_{ps}^{harped} f_{px} \sin\Psi$$

<u>Determine Transfer Lengths, Development Lengths, and f_{px}</u> (5.11.4)

The transfer length, l_{trans}, with the respect to CL bearing is found as:

$$l_{trans} = 60d_b - x_{brg} \qquad \text{(5.11.4.1)}$$

= 60(0.5 in.) – 6 in.

= 24 in. from CL bearing

The development length, l_d, may be found using the following equation:

$$l_d \geq K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \quad \text{(Eq. 5.11.4.2-1)}$$

Where:

$K = 1.6$

$$f_{pe} = \left(\frac{100 - \%\ \text{losses}}{100}\right)\left(f_{pbt}\right)$$

$$= \left(\frac{100 - 19.6}{100}\right)(201.96\ \text{ksi})$$ (see moment calculations for losses)

$= 162.38$ ksi

$d_b = 0.5$ in.

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(Eq. 5.7.3.1.1-1)}$$

Where:

$f_{pu} = 270$ ksi

$k = 0.28$ for low relaxation strands (Table 5.4.4.1-1)

$c = 5.92$ in. (see moment calculations)

$d_e = h - (C_b - e_{End})$

$= 62$ in. – (24.97 in. – 11.47 in.) = 48.5 in.

$$f_{ps} = (270\ \text{ksi})\left(1 - 0.28\frac{5.92\ \text{in.}}{48.5\ \text{in.}}\right) \quad \text{(Eq. 5.7.3.1.1-1)}$$

$= 260.8$ ksi

$$l_d \geq 1.6\left(260.8\ \text{ksi} - \frac{2}{3}(162.38\ \text{ksi})\right)(0.5\ \text{in.}) \quad \text{(Eq. 5.11.4.2-1)}$$

≥ 122.04 in.

The critical section is at x = 4.97 ft. $\left(\frac{12\text{ in.}}{\text{ft.}}\right)$ = 59.64 in. from CL bearing = l_{px}

24 in. < 59.64 in. < 122.04 in. Therefore,

$$f_{px} = f_{pe} + \frac{l_{px} - l_{trans}}{l_d - l_{trans}}\left(f_{ps} - f_{pe}\right) \quad \text{(Eq. 5.11.4.2-3)}$$

Where:

f_{ps} = 260.8 ksi

f_{pe} = 162.38 ksi

l_{px} = 59.64 in.

l_{trans}= 24 in.

d_b = 0.5 in.

$$f_{px} = 162.38\text{ ksi} + \frac{59.64\text{ in.} - 24\text{ in.}}{122.04\text{ in.} - 24\text{ in.}}\left(260.8\text{ ksi} - 162.38\text{ ksi}\right) \quad \text{(Eq. 5.11.4.2-3)}$$

$= 198.16$ ksi

A_{ps}^{harped} = (A_{strand} * # of harped strands)

$$= \left(\frac{0.153\text{ in.}^2}{\text{strand}}\right)\left(6\text{ harped strands}\right)$$

$= 0.918$ in.^{2}

$$\Psi = \arctan\left(\frac{d_{harping}}{0.4L}\right)$$

Where:

$d_{harping}$ = h_{beam} – 3 in. – dist. from top strand to bottom of beam at CL span

= 54 in. – 3 in. – 6 in.

= 45 in.

$$L = 91\text{ ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right) = 1092\text{ in.}$$

$$\Psi = \arctan\left(\frac{45\text{ in.}}{0.4(1092\text{ in.})}\right)$$

$= 5.88^\circ$

$V_p = (0.918 \text{ in.}^2)(198.16 \text{ ksi})(\sin 5.88^\circ)$

$= 18.64 \text{ k}$

Shear Resistance Due to Concrete V_c

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \qquad \text{(Eq. 5.8.3.3-3)}$$

Where:

$f'_c = 6 \text{ ksi}$

$d_v = 47.44 \text{ in.}$

$b_v = 6 \text{ in.}$

$$\beta = \frac{4.8}{1+750\varepsilon_s} \text{ (assume Art. 5.8.2.5 satisfied, then check)} \qquad \text{(Eq. 5.8.3.4.2-1)}$$

Where:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{\left(E_sA_s + E_pA_{ps}\right)} \geq 0 \qquad \text{(Eq. 5.8.3.4.2-4)}$$

Where:

$$|M_u| = (1012.1 \text{ k-ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 12145.2 \text{ k-in.}$$

Check that $|M_u| \geq |V_u - V_p|d_v$:

$d_v = 47.44 \text{ in.}$

$N_u = 0 \text{ k}$

$V_u = 217.4 \text{ k}$

$V_p = 18.64 \text{ k}$

$$|V_u - V_p|d_v = |217.4 \text{ k} - 18.64 \text{ k}|(47.44 \text{ in.})$$

$= \quad 9424.95$ k-in.

$|M_u| \geq |V_u - V_p| d_v$, use $|M_u|$.

A_{ps} = Area of prestressing steel on flexural tension side of member (in.^{2}). Determine how many strands are on flexural tension side of member:

Strand Pattern at Critical Section (Harped Strands Only)

By inspection, all of the non-harped strands are on the bottom half i.e. flexural tension side of the composite section.

To find the height of a row of strands at the critical section for shear, d_{Row}^{Crit} :

$$d_{Row}^{Crit} = d_{Row}^{End} - (x_{Crit} + x_{brg}) \tan \Psi$$

$$d_{1T}^{Crit} = 51 \text{ in.} - (59.64 \text{ in.} + 6 \text{ in.}) \tan 5.88^\circ = 44.24 \text{ in.}$$

$$d_{2T}^{Crit} = 49 \text{ in.} - (59.64 \text{ in.} + 6 \text{ in.}) \tan 5.88^\circ = 42.24 \text{ in.}$$

$$d_{3T}^{Crit} = 47 \text{ in.} - (59.64 \text{ in.} + 6 \text{ in.}) \tan 5.88^\circ = 40.24 \text{ in.}$$

The depth to the flexural tension side of the member is found as follows:

0.5(54 in. beam + 8 in. slab) = 31 in. from bottom of beam

All of the harped strands are above this point.

A_{ps} = (A_{strand} * # of strands in flexural tension side of member)

$$= \left(\frac{0.153 \text{ in.}^2}{\text{strand}}\right)(22 \text{ strands in flexural tension side of member})$$

$= 3.366$ in.^{2}

$f_{po} = 0.7 f_{pu}$

$= 0.7(270 \text{ ksi})$

$= 189 \text{ ksi}$

$E_s = 29000 \text{ ksi}$

$A_s = 0 \text{ in.}^2$

$E_p = 28500 \text{ ksi}$

$$\varepsilon_s = \frac{\left(\frac{|12145.2 \text{ k} - \text{in.}|}{47.44 \text{ in.}} + 0.5(0 \text{ k}) + |217.4 \text{ k} - 18.64 \text{ k}| - (3.366 \text{ in.}^2)(189 \text{ ksi})\right)}{(29000 \text{ ksi})(0 \text{ in.}^2) + (28500 \text{ ksi})(3.366 \text{ in.}^2)}$$

(Eq. 5.8.3.4.2-4)

$\varepsilon_s = -0.00189 \text{ in./in.}$

If ε_s is calculated as less than zero, it may be taken as zero.

$\varepsilon_s = 0 \text{ in./in.}$

$$\beta = \frac{4.8}{1 + 750(0 \text{ in./in.})} = 4.8$$ (Eq. 5.8.3.4.2-1)

$$V_c = 0.0316(4.8)\sqrt{6 \text{ ksi}}(6 \text{ in.})(47.44 \text{ in.}) = 105.75 \text{ k}$$ (Eq. 5.8.3.3-3)

Required Spacing of Transverse Reinforcement for Nominal Shear Resistance

$$s \le \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$

Where:

$$A_v = \left(\frac{0.20 \text{ in.}^2}{\text{leg}}\right)(2 \text{ legs of } \#4 \text{ stirrups})$$

$= 0.40 \text{ in.}^2$

$f_y = 60 \text{ ksi}$

$d_v = 47.44 \text{ in.}$

$\theta = 29 + 3500\varepsilon_s$ (Eq. 5.8.3.4.2-3)

$= 29 + 3500(0 \text{ in./in.})$

$= 29^\circ$

$V_u = 217.4 \text{ k}$

$\phi = 0.9$

$V_p = 18.64 \text{ k}$

$V_c = 105.75 \text{ k}$

$$s \le \frac{(0.40 \text{ in.}^2)(60 \text{ ksi})(47.44 \text{ in.})(\cot 29^\circ)}{\frac{217.4 \text{ k}}{0.9} - 18.64 \text{ k} - 105.75 \text{ k}}$$

$\le 17.5 \text{ in.}$

Maximum Permitted Spacing of Transverse Reinforcement *(5.8.2.7)*

$$v_u = \frac{|V_u - \phi V_p|}{\phi\, b_v d_v}$$ (Eq. 5.8.2.9-1)

Where:

$V_u = 217.4 \text{ k}$

$V_p = 18.64 \text{ k}$

$\phi = 0.9$

$b_v = 6 \text{ in.}$

$d_v = 47.44 \text{ in.}$

$$v_u = \frac{|217.4 \text{ k} - (0.9)(18.64 \text{ k})|}{(0.9)(6 \text{ in.})(47.44 \text{ in.})}$$

$= 0.78 \text{ ksi}$

$0.125f'_c = 0.125(6 \text{ ksi})$

$= 0.75 \text{ ksi}$

$v_u \geq 0.125f'_c$, therefore:

$$s_{max} = 0.4d_v \leq 12.0 \text{ in.} \qquad \text{(Eq. 5.8.2.7-2)}$$
$$= 0.4(47.5 \text{ in.})$$
$$= 18.99 \text{ in.} > 12 \text{ in.}$$

The maximum spacing of transverse reinforcement for strength at the critical section for shear is 12 in.

Minimum Transverse Reinforcement *(5.8.2.5)*

$$s \leq \frac{A_v f_y}{0.0316\sqrt{f'_c}\, b_v}$$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

f'_c = 6 ksi

b_v = 6 in.

$$s \leq \frac{(0.40 \text{ in.}^2)(60 \text{ ksi})}{0.0316\sqrt{6 \text{ ksi}}(6 \text{ in.})} = 51.7 \text{ in.}$$

Interface Shear Transfer Reinforcement *(5.8.4)*

The maximum spacing for interface shear reinforcement is:

$$s = \frac{12\mu A_{vf} f_y}{\frac{V_{ui}}{\phi} - cA_{cv}}$$

Where:

μ = 1.0

A_{vf} = 0.40 in.^{2}

f_y = 60 ksi

ϕ = 0.9

c = 0.28 ksi (5.8.4.3)

b_{vi} = top flange width = 20 in.

L_{vi} = 12 in.

A_{cv} = $b_{vi}L_{vi}$ (Eq. 5.8.4.1-6)

= (20 in.)(12 in.)

= 240 in.^{2}

V_{ui} = $v_{ui}A_{cv}$ (Eq. 5.8.4.2-2)

Where:

$$v_{ui} = \frac{V_u}{b_{vi}d_v}$$ (Eq. 5.8.4.2-1)

Where:

V_u = 217.4 k

b_{vi} = top flange width = 20 in.

d_v = 47.44 in.

$$v_{ui} = \frac{217.4\text{ k}}{(20\text{ in.})(47.44\text{ in.})}$$ (Eq. 5.8.4.2-1)

= 0.23 ksi

V_{ui} = (0.23 ksi)(240 in.^{2})

= 55.2 k

Check that $\frac{V_{ui}}{\phi} \leq K_1 f'_c A_{cv}$

Where:

V_{ui} = 55.2 k

ϕ = 0.9

K_1 = 0.3 (5.8.4.3)

f'_c = 6 ksi

A_{cv} = 240 in.^{2} (Eq. 5.8.4.1-6)

$$\frac{V_{ui}}{\phi} = \frac{55.2\text{ k}}{0.9} = 61.3\text{ k}$$

$K_1 f'_c A_{cv}$ = (0.3)(6 ksi)(240 in.^{2}) = 432 k > 61.3 k

Check that $\frac{V_{ui}}{\phi} \leq K_2 A_{cv}$

Where:

V_{ui} = 55.2 k

ϕ = 0.9

K_2 = 1.8 ksi (5.8.4.3)

A_{cv} = 240 in.^{2} (Eq. 5.8.4.1-6)

$$\frac{V_{ui}}{\phi} = \frac{55.2\text{ k}}{0.9} = 61.3\text{ k}$$

$K_2 A_{cv}$ = (1.8 ksi)(240 in.^{2}) = 432 k > 61.3 k

For $\frac{V_{ui}}{\phi}$, use 61.3 k.

$$s = \frac{12(1.0)(0.4\text{ in.}^2)(60\text{ ksi})}{61.3\text{ k} - (0.28\text{ ksi})(240\text{ in.}^2)} \quad \text{(Eq. i)}$$

$= -49.06$ in.

Since s < 0, no reinforcement is required for interface shear. Check the spacing required to satisfy $\frac{1.33V_{ui}}{\phi}$:

$$s = \frac{12(1.0)(0.4\text{ in.}^2)(60\text{ ksi})}{1.33(61.3\text{ k}) - (0.28\text{ ksi})(240\text{ in.}^2)} \quad \text{(Eq. iii)}$$

$$= 20.1\text{ in.}$$

Check the minimum spacing required by Equation ii:

$$s = \frac{12A_{vf}f_y}{0.05A_{cv}} \quad \text{(Eq. ii)}$$

$$= \frac{12(0.4\text{ in.}^2)(60\text{ ksi})}{0.05(240\text{ in.}^2)}$$

$$= 24\text{ in.}$$

The required spacing for interface shear reinforcement at the critical point for shear is 24 in.

Determine Controlling Transverse Reinforcement Spacing

The required reinforcement spacing for strength is 17.5 in.
The maximum permitted reinforcement spacing is 12 in.
The minimum reinforcement area is based on a spacing of 24 in.
The required reinforcement spacing for interface shear transfer is 24 in.

The controlling spacing at the critical section for shear is 12 in.

Longitudinal Reinforcement *(5.8.3.5)*

Critical Section*:*

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \quad \text{(Eq. 5.8.3.5-2)}$$

Where:

A_s = 0 in.^{2}

f_y = 60 ksi

A_{ps} = 3.366 in.^{2}

f_{ps} = 260.8 ksi

$|V_u|$ = 217.4 k

ϕ_v = 0.9

V_s = $\dfrac{A_v f_y d_v \cot\theta}{s}$, not to be taken as larger than $\dfrac{|V_u|}{\phi_v}$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

d_v = 47.44 in.

θ = 29°

s = 12 in.

$$\frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.40 \text{ in.}^2)(60 \text{ ksi})(47.44 \text{ in.})(\cot 29^\circ)}{12 \text{ in.}} = 171.17 \text{ k}$$

$$\frac{|V_u|}{\phi_v} = \frac{|217.4 \text{ k}|}{0.9} = 241.6 \text{ k}$$

V_s = 171.17 k

V_p = 18.64 k

θ = 29°

$$A_s f_y + A_{ps} f_{ps} = (0 \text{ in.}^2)(60 \text{ ksi}) + (3.366 \text{ in.}^2)(260.8 \text{ ksi}) = 877.9 \text{ k}$$

$$\left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p \right) \cot\theta =$$

$= (241.6\text{ k} - 0.5(171.17\text{ k}) - 18.64\text{ k})(\cot 29^\circ) = 247.8\text{ k}$

877.9 k > 247.8 k OK

Face of Abutment:

From computer software, $|V_u|$ has been calculated to be 233.2 k at the abutment face.

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad \text{(Eq. 5.8.3.5-2)}$$

Where:

A_s = 0 in.^{2}

f_y = 60 ksi

A_{ps} = 3.366 in.^{2}

f_{ps} = 260.8 ksi

$|V_u|$ = 233.2 k

ϕ_v = 0.9

V_s = $\frac{A_v f_y d_v \cot\theta}{s}$, not to be taken as larger than $\frac{|V_u|}{\phi_v}$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

d_e = $h - (C_b - e_{Trans})$

h = 54 in. beam + 8 in. slab = 62 in.

C_b = 24.97 in. (see Bridge Manual Table 3.4.4.2-1)

e_{Trans} = Eccentricity of strand group at critical section

$= e_{End} + \left(\frac{x_{Trans}}{0.4L_{beam}} \right)(e_{Center} - e_{End})$

$= 11.470\text{ in.} + \left(\frac{24\text{ in.}}{0.4(1092\text{ in.})} \right)(21.113\text{ in.} - 11.470\text{ in.})$

= 12 in.

d_e = 62 in. – (24.97 in. – 12 in.)

= 49.03 in.

d_v = d_e – 0.5a

= 49.03 in. – 0.5(5.03 in.)

= 46.52 in., which controls over 0.9d_e = 44.13 in. and 0.72h = 44.64 in.

θ = 29°

s = 12 in.

$$\frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.40\text{ in.}^2)(60\text{ ksi})(46.52\text{ in.})(\cot 29^\circ)}{12\text{ in.}} = 167.8\text{ k}$$

$$\frac{|V_u|}{\phi_v} = \frac{|233.2\text{ k}|}{0.9} = 259.1\text{ k}$$

V_s = 167.8 k

V_p = $A_{ps}^{harped} f_{px} \sin\Psi$

$$f_{px} = \frac{f_{pe} l_{px}}{60 d_b} \qquad \text{(Eq. 5.11.4.2-2)}$$

$$= \frac{(162.38\text{ ksi})(21\text{ in.})}{60(0.5\text{ in.})}$$

= 113.67 ksi

V_p = $(0.918\text{ in.}^2)(113.67\text{ ksi})\sin 5.88^\circ$

= 10.7 k

θ = 29°

$$A_s f_y + A_{ps} f_{ps} = (0\text{ in.}^2)(60\text{ ksi}) + (3.366\text{ in.}^2)(261.05\text{ ksi}) = 878.7\text{ k}$$

$$\left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p\right)\cot\theta =$$

$$= (259.1\text{k} - 0.5(167.8\text{ k}) - 10.7\text{ k})(\cot 29^\circ)$$

= 296.8 k

878.7 k > 296.8 k OK

Check Calculated Capacity at the Critical Section Against Eq. 5.8.3.3-2

V_n shall be the lesser of $V_n = V_c + V_s + V_p$ and

$$V_n = 0.25f'_c b_v d_v + V_p \quad \text{(Eq. 5.8.3.3-2)}$$

Where:

f'_c = 6 ksi

b_v = 6 in.

d_v = 47.44 in.

V_p = 18.64 k

V_n = 0.25(6 ksi)(6 in.)(47.44 in.) + 18.64 k = 445.6 k

$V_c + V_s + V_p$ = 105.75 k + 171.17 k + 18.64 k = 295.56 k

445.6 k > 295.56 k

Equation 5.8.3.3-2 does not control. The controlling spacing for strength at the critical section for shear is based on $V_c + V_s + V_p$.

Tabulated Required Maximum Spacings

Point along span	x (ft.)	s (in.) 5.8.3.3-4	s (in.) 5.8.2.5-1	s (in.) 5.8.2.7	s (in.) Eq.'s i-iii	s (in.) Governing
Critical Pt.	4.97	17.5	51.7	12.0	20.2	12.0
0.1	9.00	22.9	51.7	24.0	N/A	22.9
0.2	18.00	13.9	51.7	24.0	N/A	13.9
0.3	27.00	30.4	51.7	24.0	N/A	24.0
0.4	36.00	34.4	51.7	24.0	N/A	24.0
0.5	45.00	140.4	51.7	24.0	N/A	24.0
0.6	54.00	34.4	51.7	24.0	N/A	24.0
0.7	63.00	30.4	51.7	24.0	N/A	24.0
0.8	72.00	13.9	51.7	24.0	N/A	13.9
0.9	81.00	22.9	51.7	24.0	N/A	22.9
Critical Pt.	85.03	17.5	51.7	12.0	20.2	12.0

Example 2

140 ft., 100 ft., 2 span, 72 inch PPC Bulb T-beam, 9 beam lines, 4.5 ft. beam spacing, 8 in. deck, F-shape barrier, 50 pounds per square foot future wearing surface, no skew, 3 design lanes and HL-93 loading on integral abutments and a fixed pile bent pier. It should be noted that the non-composite span lengths were conservatively set equal to the composite span lengths for simplicity.

General Data

Design Code	=	LRFD (simplified live load distribution)
Shear Design Method	=	Simplified Shear Design Method
Span 1 length	=	140 ft.
Span 2 length	=	100 ft.
Beam section	=	72 in. PPC Bulb T-beam
Beam spacing	=	4.5 ft.
Number of beams	=	9
Deck thickness	=	8 in.
Estimated camber	=	1 in.
Average fillet	=	1.0 in. (based on ½ in. min. fillet and estimated camber)
Parapet	=	0.45 k/ft.
FWS	=	50 psf
Relative Humidity	=	70 %
Strands	=	½ in. diameter – 270 ksi low relaxation strands
Skew	=	0 degrees

Live Load Data

Loading	=	HL-93
IM	=	1.33 (HL-93); 1.15 (fatigue truck)

N_L = 3

Trial Strand Patterns

Span 1:

Select strand pattern 38DMH from planning charts in BM Section 2.3.6.1.3.

$A_{ps} = \quad 38(0.153 \text{ in.}^2) = \quad 5.814 \text{ in.}^2$

Span 2:

Select strand pattern 22SU from planning charts in BM Section 2.3.6.1.3. It should be noted that since span 1 requires high strength concrete use high strength concrete in span 2.

$A_{ps} = \quad 22(0.153 \text{ in.}^2) = \quad 3.366 \text{ in.}^2$

Materials

Precast Concrete Beam

f'_c	=	7.0 ksi
f'_{ci}	=	6.0 ksi
f_{pbt}	=	201.96 ksi
f_{pu}	=	270.0 ksi
Fi/strand	=	30.9 kips

Cast in Place Concrete Deck

f'_c	=	3.5 ksi
f_y	=	60.0 ksi

Section Properties

Modulus of Elasticity

$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c}$ (Eq. 5.4.2.4-1)

$E_{ci} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ci}}$ (Eq. 5.4.2.4-1)

$E_c \text{ (deck)} = 33{,}000(1.0)(0.15 \text{ k/ft.})^{1.5} \sqrt{3.5 \text{ ksi}} = 3587 \text{ ksi}$

$E_{ci} \text{ (beam)} = 33{,}000(1.0)(0.15 \text{ k/ft.})^{1.5} \sqrt{6.0 \text{ ksi}} = 4696 \text{ ksi}$

E_c (beam) = $33{,}000(1.0)(0.15\ \text{k/ft.})^{1.5}\sqrt{7.0\ \text{ksi}}$ = 5072 ksi

E_p (strand) = 28500 ksi (5.4.4.2)

E_s (reinf.) = 29000 ksi (5.4.3.2)

<u>Non-Composite</u> (Beam only)

A = 767.0 in.^{2}

I = 545894 in.^{4}

S_b = 14915 in.^{3} C_b = 36.60 in.

S_t = 15421 in.^{3} C_t = 35.40 in.

<u>Composite</u>

Modular Ratio:

$$n = \frac{E_c(\text{deck})}{E_c(\text{beam})} = \frac{3587\ \text{ksi}}{5072\ \text{ksi}} = 0.71$$

Effective Flange Width: (4.6.2.6)

Dist. C. to C. of beams = 4.5 ft.(12 in./ft.) = 54 in.

Compute composite section properties:

		A	y	Ay
Slab	0.71(54 in.)(8 in.)	306.7 in.^{2}	76.00 in.	23309 in.^{3}
Beam		<u>767.0 in.^{2}</u>	36.60 in.	<u>28072 in.^{3}</u>
		1073.7 in.^{2}		51381 in.^{3}

$$y_b = \frac{51381\ \text{in.}^3}{1073.7\ \text{in.}^2} = 47.85\ \text{in.}$$

y_t = 72 in. – 47.85 in. = 24.15 in.

	Io	A	d	Ad^2	I’
Slab	1636 in.^{4}	306.7 in.^{2}	28.15 in.	243036 in.^{4}	244672 in.^{4}
Beam	545894 in.^{4}	767.0 in.^{2}	11.25 in.	97073 in.^{4}	<u>642967 in.^{4}</u>
				I’ =	887639 in.^{4}

$$S'_b = \frac{887639 \text{ in.}^4}{47.85 \text{ in.}} = 18550 \text{ in.}^3$$

$$S'_t = \frac{887639 \text{ in.}^4}{24.15} = 36755 \text{ in.}^3$$

Distribution Factors *(BM Section 3.3.1)*

C = 1.15

Span 1:

<u>Moment</u>

$$g_1 = 0.06 + C\left(\frac{S}{14.0}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3} = 0.06 + 1.15\left(\frac{4.5 \text{ ft.}}{14.0}\right)^{0.4}\left(\frac{4.5 \text{ ft.}}{140 \text{ ft.}}\right)^{0.3}$$

$$= 0.320$$

$$g_m = 0.075 + C\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2} = 0.075 + 1.15\left(\frac{4.5 \text{ ft.}}{9.5}\right)^{0.6}\left(\frac{4.5 \text{ ft.}}{140 \text{ ft.}}\right)^{0.2}$$

$= 0.444$ <= Controls

<u>Moment (fatigue loading)</u>

$$g_1 \text{ (fatigue)} = \frac{g_1}{m} = \frac{0.320}{1.2} = 0.267$$

<u>Shear and Reaction</u>

$$g_1 = 0.36 + \frac{S}{25.0} = 0.36 + \frac{4.5 \text{ ft.}}{25.0} = 0.540$$

$$g_m = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} = 0.2 + \left(\frac{4.5 \text{ ft.}}{12}\right) - \left(\frac{4.5 \text{ ft.}}{35}\right)^{2.0} = 0.558$$ <= Controls

$$\text{Skew correction} = 1 + 0.2\tan(\theta)$$

$$= 1 + 0.2\tan(0)$$

$$= 1.000$$

Deflection

$$g \text{ (deflection)} = m\left(\frac{N_L}{N_b}\right) = 0.85\left(\frac{3}{9}\right) = .283$$

Span 2:

Moment

$$g_1 = 0.06 + 1.15\left(\frac{4.5\text{ ft.}}{14.0}\right)^{0.4}\left(\frac{4.5\text{ ft.}}{100\text{ ft.}}\right)^{0.3}$$

$$= 0.348$$

$$g_m = 0.075 + 1.15\left(\frac{4.5\text{ ft.}}{9.5}\right)^{0.6}\left(\frac{4.5\text{ ft.}}{100\text{ ft.}}\right)^{0.2}$$

$$= 0.470 \quad \Leftarrow \text{Controls}$$

Moment (fatigue loading)

$$g_1 \text{ (fatigue)} = \frac{0.348}{1.2} = 0.290$$

Shear and Reaction

Same as Span 1

Deflection

Same as Span 1

Pier:

Use average span length of the two adjacent spans to the pier.

$$L \text{ (average)} = \frac{(140\text{ ft.} + 100\text{ ft.})}{2} = 120\text{ ft.}$$

Moment

$$g_1 = 0.06 + 1.15\left(\frac{4.5 \text{ ft.}}{14.0}\right)^{0.4}\left(\frac{4.5 \text{ ft.}}{120 \text{ ft.}}\right)^{0.3} = 0.333$$

$$g_m = 0.075 + 1.15\left(\frac{4.5 \text{ ft.}}{9.5}\right)^{0.6}\left(\frac{4.5 \text{ ft.}}{120 \text{ ft.}}\right)^{0.2} = 0.456 \quad \text{<= Controls}$$

Moment (fatigue loading)

$$g_1 \text{ (fatigue)} = \frac{0.333}{1.2} = 0.278$$

Shear and Reaction

Same as Span 1 and Span 2

Dead Loads

Non-Composite

DC1:

Beam = 0.799 k/ft.

Slab $\left(0.15 \text{ k/ft.}^3\right)\left(\frac{8 \text{ in.}}{12 \text{ in./ft.}}\right)\left(4.5 \text{ ft.}\right)$ = 0.450 k/ft.

Fillet $\left(0.15 \text{ k/ft}^3\right)\left(\frac{1.0 \text{ in.}}{12 \text{ in./ft.}}\right)\left(\frac{42 \text{ in.}}{12 \text{ in./ft.}}\right)$ = 0.044 k/ft.

DW1:

No non-composite dead loads in this category.

Composite

DC2:

Parapets $\frac{\left(0.45 \text{ k/ft.}\right)\left(2 \text{ parapets}\right)}{9 \text{ beams}}$ = 0.100 k/ft.

DW2:

FWS $\left(0.050 \text{ k/ft.}^2\right)\left(4.5 \text{ ft.}\right)$ = 0.225 k/ft.

Maximum Unfactored Distributed Moments

For this example the maximum moments were calculated at 0.4 and 0.5 of span 1 and 0.5 and 0.6 of span 2 since it is uncertain which point will control. All moments are generated by computer software.

Span 1:

Non-Composite

DC1:

@0.4L M_{DC1} = 3041 k-ft.

@0.5L M_{DC1} = 3168 k-ft.

DW1:

No non-composite DW1 loads.

Composite

DC2:

@0.4L M_{DC2} = 157 k-ft.

@0.5L M_{DC2} = 148 k-ft.

DW2:

@0.4L M_{DW2} = 354 k-ft.

@0.5L M_{DW2} = 332 k-ft.

LL+IM:

@0.4L M_{LL+IM} = 1548 k-ft.

@0.5L M_{LL+IM} = 1505 k-ft.

@0.4L M_{FL+IM} = 478 k-ft.

@0.5L M_{FL+IM} = 462 k-ft.

Pier:

Non-Composite

No non-composite loads.

Composite

DC2:

M_{DC2} = -195 k-ft.

DW2:

M_{DW2} = -439 k-ft.

LL+IM:

M_{LL+IM} = -1420 k-ft.

M_{FL+IM} = -339 k-ft.

Span 2:

Non-Composite

DC1:

@0.5L M_{DC1} = 1616 k-ft.

@0.6L M_{DC1} = 1552 k-ft.

DW1:

No non-composite DW1 loads.

Composite

DC2:

@0.5L M_{DC2} = 28 k-ft.

@0.6L M_{DC2} = 42 k-ft.

DW2:

@0.5L M_{DW2} = 62 k-ft.

@0.6L M_{DW2} = 95 k-ft.

LL+IM:

@0.5L M_{LL+IM} = 1085 k-ft.

@0.6L M_{LL+IM} = 1095 k-ft.

@0.5L M_{FL+IM} = 348 k-ft.

@0.6L M_{FL+IM} = 359 k-ft.

Lifting Loop Design

Span 1:

Beam length = 140.2 ft.

Total beam weight= $(0.799\ k/ft.)(140.2\ ft.)$ = 112 kips

From the charts: 2 - 5 strand lifting loops required each end at 4.5 ft. and 8.5 ft.

Span 2:

Beam length = 100.2 ft.

Total beam weight= $(0.799\ k/ft.)(100.2\ ft.)$ = 80 kips

From the charts: 2 - 3 strand lifting loops required each end at 4.5 ft. and 8.5 ft.

Strand Eccentricities

The length of beam was used instead of the length of span when calculating the eccentricity along draped strands.

Span 1:

Eccentricity at center:

Row	Number (N)	m	N(m)
1	12	2 in.	24 in.
2	12	4 in.	48 in.
3	4	6 in.	24 in.
4	2	8 in.	16 in.
5	2	10 in.	20 in.
6	2	12 in.	24 in.
7	2	14 in.	28 in.
8	2	16 in.	32 in.
Total	38		216 in.

C.G. of strands measured from bottom = $\frac{216\ in.}{38}$ = 5.68 in.

$e = C_b - 5.68\ in. = 36.60\ in. - 5.68\ in. = 30.92\ in.$

Eccentricity at end:

Row	Number (N)	m	N(m)
1	10	2 in.	20 in.
2	10	4 in.	40 in.
3	2	6 in.	12 in.
8T	2	55 in.	110 in.
7T	2	57 in.	114 in.
6T	2	59 in.	118 in.
5T	2	61 in.	122 in.
4T	2	63 in.	126 in.
3T	2	65 in.	130 in.
2T	2	67 in.	134 in.
1T	2	69 in.	138 in.
Total	38		1064 in.

C.G. of strands measured from bottom $= \dfrac{1064 \text{ in.}}{38} = 28.00$ in.

$e = C_b - 28.00 \text{ in.} = 36.60 \text{ in.} - 28.00 \text{ in.} = 8.6$ in.

Eccentricity at transfer point:

Transfer length = 60 strand diameters from beam end, see 5.11.4.1.

$$\text{Transfer length} = \frac{60(0.5\text{ in.})}{(12\text{ in./ft.})} = 2.5\text{ ft. from beam end}$$

Row	Number (N)	m	N(m)
1	10	2 in.	20 in.
2	10	4 in.	40 in.
3	2	6 in.	12 in.
8T	2	52.6 in.	105.2 in.
7T	2	54.6 in.	109.2 in.
6T	2	56.6 in.	113.2 in.
5T	2	58.6 in.	117.2 in.
4T	2	60.6 in.	121.2 in.
3T	2	62.6 in.	125.2 in.
2T	2	64.6 in.	129.2 in.
1T	2	66.6 in.	133.2 in.
Total	38		1025.6 in.

$$\text{C.G. of strands measured from bottom} = \frac{1025.6\text{ in.}}{38} = 26.99\text{ in.}$$

$$e = C_b - 26.99\text{ in.} = 36.60\text{ in.} - 26.99\text{ in.} = 9.61\text{ in.}$$

Eccentricity at center of lifting loops (use average of 4.5 ft. and 8.5 ft. = 6.5 ft.):

Row	Number (N)	m	N(m)
1	10	2 in.	20 in.
2	10	4 in.	40 in.
3	2	6 in.	12 in.
8T	2	48.9 in.	97.8 in.
7T	2	50.9 in.	101.8 in.
6T	2	52.9 in.	105.8 in.
5T	2	54.9 in.	109.8 in.
4T	2	56.9 in.	113.8 in.
3T	2	58.9 in.	117.8 in.
2T	2	60.9 in.	121.8 in.
1T	2	62.9 in.	125.8 in.
Total	38		966.4 in.

$$\text{C.G. of strands measured from bottom} = \frac{966.4\text{ in.}}{38} = 25.43\text{ in.}$$

$$e = C_b - 25.43\text{ in.} = 36.60\text{ in.} - 25.43\text{ in.} = 11.17\text{ in.}$$

Span 2:

Eccentricity at all locations:

Row	Number (N)	m	N(m)
1	6	2 in.	12 in.
2	4	4 in.	16 in.
3	4	6 in.	24 in.
4	2	8 in.	16 in.
5	2	10 in.	20 in.
6	2	12 in.	24 in.
7	2	14 in.	28 in.
Total	22		140 in.

$$\text{C.G. of strands measured from bottom} = \frac{140\text{ in.}}{22} = 6.36\text{ in.}$$

$$e = C_b - 6.36\text{ in.} = 36.60\text{ in.} - 6.36\text{ in.} = 30.24\text{ in.}$$

Total Loss of Prestress

Span 1:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.5.1-1)

Instantaneous Losses (due to elastic shortening):

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ (Eq. 5.9.5.2.3a-1)

Assume F_t equals 90 percent of F_i for first iteration:

$$F_i = A_{ps}(f_{pbt}) = (5.814 \text{ in.}^2)(201.96 \text{ ksi}) = 1174 \text{ kips}$$

$$F_t = 0.9(F_i) = 0.9(1174 \text{ kips}) = 1057 \text{ kips}$$

$$M_b \text{ (beam)} = \frac{(0.799 \text{ k/ft.})(140 \text{ ft.})^2}{8} = 1958 \text{ k-ft.}$$

$$f_{cgp} = \frac{F_t}{A} + \frac{F_t e^2}{I} - \frac{M_b(12)e}{I}$$

$$= \frac{1057 \text{ kips}}{767 \text{ in.}^2} + \frac{(1057 \text{ kips})(30.92 \text{ in.})^2}{545894 \text{ in.}^4} - \frac{(1958 \text{ k} - \text{ft.})(12 \text{ in./ft.})(30.92 \text{ in.})}{545894 \text{ in.}^4}$$

$$= 1.90 \text{ ksi}$$

Calculate Δf_{pES}:

$$\Delta f_{pES} = \frac{28500 \text{ ksi}}{4696 \text{ ksi}}(1.90 \text{ ksi}) = 11.53 \text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} = \frac{(201.96 \text{ ksi} - 11.53 \text{ ksi})}{201.96 \text{ ksi}} = 0.94 > 0.90$$

Recalculate

Assume F_t equals 94 percent of F_i for second iteration:

$$F_t = 0.94(1174\text{ kips}) = 1104\text{ kips}$$

$$f_{cgp} = \frac{1104\text{ kips}}{767\text{ in.}^2} + \frac{(1104\text{ kips})(30.92\text{ in.})^2}{545894\text{ in.}^4} - \frac{(1958\text{ k} - \text{ft.})(12\text{ in./ft.})(30.92\text{ in.})}{545894\text{ in.}^4}$$

$$= 2.04\text{ ksi}$$

Calculate Δf_{pES}:

$$\Delta f_{pES} = \frac{28500\text{ ksi}}{4696\text{ ksi}}(2.04\text{ ksi}) = 12.38\text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} = \frac{(201.96\text{ ksi} - 12.38\text{ ksi})}{201.96\text{ ksi}} = 0.94 \quad \text{Ok}$$

Time Dependent Losses:

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(Eq. 5.9.5.3-1)}$$

In which:

$$\gamma_h = 1.7 - 0.01H = 1.7 - 0.01(70) = 1.0 \qquad \text{(Eq. 5.9.5.3-2)}$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})} = \frac{5}{(1 + 6\text{ ksi})} = 0.714 \qquad \text{(Eq. 5.9.5.3-3)}$$

$$\Delta f_{pR} = 2.4\text{ ksi}$$

$$\Delta f_{pLT} = 10.0\frac{(201.96\text{ ksi})(5.814\text{ in.}^2)}{767\text{ in.}^2}(1.0)(0.714) + 12.0(1.0)(0.714) + 2.4\text{ ksi}$$

$$= 21.90\text{ ksi}$$

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 12.38\text{ ksi} + 21.90\text{ ksi} = 34.28\text{ ksi}$$

$$\%\text{Loss} = \frac{34.28\text{ ksi}}{201.96\text{ ksi}} = 17.0\ \%$$

Span 2:

$\Delta f_{pT} \quad = \quad \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.5.1-1)

Instantaneous Losses (due to elastic shortening):

$$\Delta f_{pES} \quad = \quad \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{(Eq. 5.9.5.2.3a-1)}$$

Assume F_t equals 90 percent of F_i for first iteration:

$F_i \quad = \quad A_{ps}(f_{pbt}) = \quad 3.366 \text{ in.}^2 (201.96 \text{ ksi}) \quad = \quad 680 \text{ kips}$

$F_t \quad = \quad 0.9(F_i) \; = \quad 0.9(680 \text{ kips}) \quad = \quad 612 \text{ kips}$

$$M_b \text{ (beam)} \quad = \quad \frac{(0.799 \text{ k/ft.})(100 \text{ ft.})^2}{8} \quad = \quad 999 \text{ k-ft.}$$

$$f_{cgp} = \quad \frac{F_t}{A} + \frac{F_t e^2}{I} - \frac{M_b (12) e}{I}$$

$$= \quad \frac{612 \text{ kips}}{767 \text{ in.}^2} + \frac{(612 \text{ kips})(30.24 \text{ in.})^2}{545894 \text{ in.}^4} - \frac{(999 \text{ k} - \text{ft.})(12 \text{ in./ft.})(30.24 \text{ in.})}{545894 \text{ in.}^4}$$

$= \quad 1.16 \text{ ksi}$

Calculate Δf_{pES}:

$$\Delta f_{pES} \quad = \quad \frac{28500 \text{ ksi}}{4696 \text{ ksi}} (1.16 \text{ ksi}) \quad = \quad 7.04 \text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} \quad = \quad \frac{(201.96 \text{ ksi} - 7.04 \text{ ksi})}{201.96 \text{ ksi}} \quad = \quad 0.97 \quad > \quad 0.90$$

Recalculate

Assume F_t equals 96 percent of F_i for second iteration:

$F_t \quad = \quad 0.96(680 \text{ kips}) \quad = \quad 653 \text{ kips}$

$$f_{cgp} = \quad \frac{653 \text{ kips}}{767 \text{ in.}^2} + \frac{(653 \text{ kips})(30.24 \text{ in.})^2}{545894 \text{ in.}^4} - \frac{(999 \text{ k} - \text{ft.})(12 \text{ in./ft.})(30.24 \text{ in.})}{545894 \text{ in.}^4}$$

$= \quad 1.28 \text{ ksi}$

Calculate Δf_{pES}:

$$\Delta f_{pES} = \frac{28500 \text{ ksi}}{4696 \text{ ksi}}(1.28 \text{ ksi}) = 7.77 \text{ ksi}$$

Check Assumption:

$$\frac{(f_{pbt} - \Delta f_{pES})}{f_{pbt}} = \frac{(201.96 \text{ ksi} - 7.77 \text{ ksi})}{201.96 \text{ ksi}} = 0.96 \quad \text{Ok}$$

Time Dependent Losses:

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(Eq. 5.9.5.3-1)}$$

In which:

$$\gamma_h = 1.7 - 0.01H = 1.7 - 0.01(70) = 1.0 \qquad \text{(Eq. 5.9.5.3-2)}$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})} = \frac{5}{(1 + 6 \text{ ksi})} = 0.714 \qquad \text{(Eq. 5.9.5.3-3)}$$

$\Delta f_{pR} = 2.4$ ksi

$$\Delta f_{pLT} = 10.0\frac{(201.96 \text{ ksi})(3.366 \text{ in.}^2)}{767 \text{ in.}^2}(1.0)(0.714) + 12.0(1.0)(0.714) + 2.4 \text{ ksi}$$

$$= 17.30 \text{ ksi}$$

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 7.77 \text{ ksi} + 17.30 \text{ ksi} = 25.07 \text{ ksi}$$

$$\%\text{Loss} = \frac{25.07 \text{ ksi}}{201.96 \text{ ksi}} = 12.4 \%$$

Stress Limits for Concrete

Temporary stresses (5.9.4.1)

Compression:

$$0.60f'_{ci} = 0.60(6.0\text{ ksi}) = 3.60\text{ ksi}$$

Tension:

$$0.24\sqrt{f'_{ci}} = 0.24\sqrt{6.0\text{ ksi}} = 0.588\text{ ksi}$$

Service stresses after losses (5.9.4.2)

Compression (For Service I load combination):

$$0.60\phi_w f'_c = 0.60(1.0)(7.0\text{ ksi}) = 4.20\text{ ksi} \quad \text{(a)}$$

$$0.45f'_c = 0.45(7.0\text{ ksi}) = 3.15\text{ ksi} \quad \text{(b)}$$

Tension (For Service III load combination):

$$0.19\sqrt{f'_c} = 0.19\sqrt{7.0\text{ ksi}} = 0.503\text{ ksi}$$

Fatigue stresses after losses (5.5.3.1)

Compression (For Fatigue I load combination):

$$0.40f'_c = 0.40(7.0\text{ ksi}) = 2.80\text{ ksi}$$

Tension limit for determination of cracked verse uncracked section:

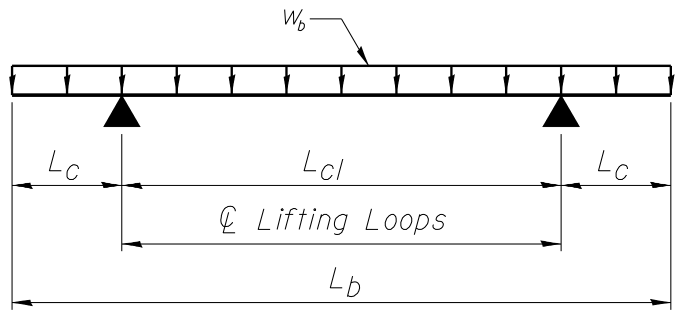
$$\text{Uncracked} \le 0.095\sqrt{f'_c} \le \text{Cracked}$$

$$0.095\sqrt{7.0\text{ ksi}} = 0.251\text{ ksi}$$

Check Temporary Stresses

Span 1:

Calculate Dead Load Moments for Determining Temporary Stresses



@ Lifting Loops:

$$M_{bts} = \frac{w_b (L_c)^2}{2} = \frac{(0.799\ k/ft.)(6.5\ ft.)^2}{2} = 17\ \text{k-ft.}$$

@ Harping Point (0.4 L_b):

$$M_{bts} = \frac{3w_b L_b^2}{25} - \frac{w_b L_b L_c}{2}$$

$$= \frac{3(0.799\ k/ft.)(140.2\ ft.)^2}{25} - \frac{(0.799\ k/ft.)(140.2\ ft.)(6.5\ ft.)}{2}$$

$$= 1521\ \text{k-ft.}$$

Prestress Force Immediately after Transfer

$$F_t = A_{ps}(f_{pbt} - \Delta f_{pES}) = 5.814\ in.^2(201.96\ ksi - 12.38\ ksi) = 1102\ kips$$

Temporary Stresses

@ Lifting Loops:

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} - \frac{M_{bts}(12)}{S_t}$$

$$= \frac{1102\ kips}{767\ in.^2} - \frac{(1102\ kips)(11.17\ in.)}{15421\ in.^3} - \frac{(17\ k-ft.)(12\ in./ft.)}{15421\ in.^3}$$

$= 0.625$ ksi (comp.) in compression Ok

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} + \frac{M_{bts}(12)}{S_b}$$

$$= \frac{1102\text{ kips}}{767\text{ in.}^2} + \frac{(1102\text{ kips})(11.17\text{ in.})}{14915\text{ in.}^3} + \frac{(17\text{k} - \text{ft.})(12\text{ in./ft.})}{14915\text{ in.}^3}$$

$= 2.276$ ksi (comp.) $\leq$ 3.600 ksi Ok

@ Harping Point (0.4 L_b):

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} + \frac{M_{bts}(12)}{S_t}$$

$$= \frac{1102\text{ kips}}{767\text{ in.}^2} - \frac{(1102\text{ kips})(30.92\text{ in.})}{15421\text{ in.}^3} + \frac{(1521\text{ k} - \text{ft.})(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$= 0.411$ ksi (comp.) in compression Ok

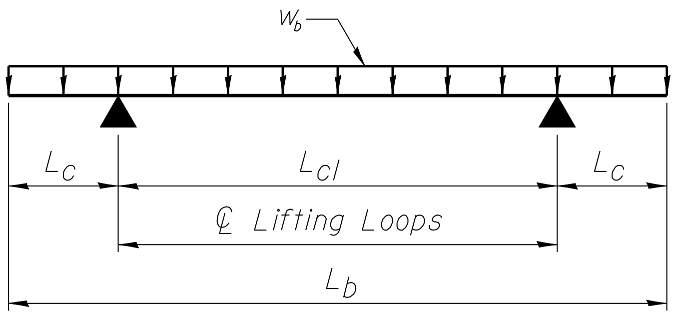
$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} - \frac{M_{bts}(12)}{S_b}$$

$$= \frac{1102\text{ kips}}{767\text{ in.}^2} + \frac{(1102\text{ kips})(30.92\text{ in.})}{14915\text{ in.}^3} - \frac{(1521\text{ k} - \text{ft.})(12\text{ in./ft.})}{14915\text{ in.}^3}$$

$= 2.498$ ksi (comp.) $\leq$ 3.600 ksi Ok

Span 2:

Calculate Dead Load Moments for Determining Temporary Stresses



@ Lifting Loops:

$$M_{bts} = \frac{w_b (L_c)^2}{2} = \frac{(0.799\ k/ft.)(6.5\ ft.)^2}{2} = 17\ k\text{-}ft.$$

@ Center:

$$M_{bts} = \frac{w_b (L_b)^2}{8} - \frac{w_b L_b L_c}{2}$$

$$= \frac{(0.799\ k/ft.)(100.2\ ft.)^2}{8} - \frac{(0.799\ k/ft.)(100.2\ ft.)(6.5\ ft.)}{2}$$

$$= 743\ k\text{-}ft.$$

Prestress Force Immediately after Transfer

$$F_t = A_{ps}(f_{pbt} - \Delta f_{pES}) = 3.366\ in.^2 (201.96\ ksi - 7.77\ ksi) = 654\ kips$$

Temporary Stresses

@ Lifting Loops:

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} - \frac{M_{bts}(12)}{S_t}$$

$$= \frac{654\ kips}{767\ in.^2} - \frac{(654\ kips)(30.24\ in.)}{15421\ in.^3} - \frac{(17\ k-ft.)(12\ in./ft.)}{15421\ in.^3}$$

$$= 0.443\ ksi\ (tension) \leq 0.588\ ksi \quad Ok$$

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} + \frac{M_{bts}(12)}{S_b}$$

$$= \frac{654\text{ kips}}{767\text{ in.}^2} + \frac{(654\text{ kips})(30.24\text{ in.})}{14915\text{ in.}^3} + \frac{(17\text{ k}-\text{ft.})(12\text{ in./ft.})}{14915\text{ in.}^3}$$

= 2.192 ksi (comp.) ≤ 3.600 ksi Ok

@ Center:

$$f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} + \frac{M_{bts}(12)}{S_t}$$

$$= \frac{654\text{ kips}}{767\text{ in.}^2} - \frac{(654\text{ kips})(30.24\text{ in.})}{15421\text{ in.}^3} + \frac{(743\text{ k}-\text{ft.})(12\text{ in./ft.})}{15421\text{ in.}^3}$$

= 0.148 ksi (comp.) in compression Ok

$$f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} - \frac{M_{bts}(12)}{S_b}$$

$$= \frac{654\text{ kips}}{767\text{ in.}^2} + \frac{(654\text{ kips})(30.24\text{ in.})}{14915\text{ in.}^3} - \frac{(743\text{ k}-\text{ft.})(12\text{ in./ft.})}{14915\text{ in.}^3}$$

= 1.581 ksi (comp.) ≤ 3.600 ksi Ok

Design Positive Moment Region

Check Service Stresses after Losses

Span 1:

Prestress Force after Losses:

$$F_s = A_{ps}(f_{pbt} - \Delta f_{pT}) = (5.814\text{ in.}^2)(201.96\text{ ksi} - 34.28\text{ ksi}) = 975\text{ kips}$$

Service Stresses:

@ 0.4 span 1:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t^{'}} \quad \text{(a)}$$

$$= \frac{975\text{ kips}}{767\text{ in.}^2} - \frac{(975\text{ kips})(30.92\text{ in.})}{15421\text{ in.}^3} + \frac{(3041\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$$+ \frac{(157\text{ k}-\text{ft.} + 354\text{ k}-\text{ft.} + 1548\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3}$$

= 2.355 ksi (comp.) ≤ 4.200 ksi Ok

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \qquad \text{(b)}$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} - \frac{(975 \text{ kips})(30.92 \text{ in.})}{15421 \text{ in.}^3} + \frac{(3041 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ \frac{(157 \text{ k} - \text{ft.} + 354 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

$= 1.849$ ksi (comp.) $\leq$ 3.150 ksi Ok

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} + \frac{(975 \text{ kips})(30.92 \text{ in.})}{14915 \text{ in.}^3} - \frac{(3041 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{14915 \text{ in.}^3}$$

$$- \frac{(157 \text{ k} - \text{ft.} + 354 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3} - 0.8\frac{(1548 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3}$$

$= 0.286$ ksi (tension) $\leq$ 0.503 ksi Ok

@ 0.5 span 1:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t^{'}} \qquad \text{(a)}$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} - \frac{(975 \text{ kips})(30.92 \text{ in.})}{15421 \text{ in.}^3} + \frac{(3168 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ \frac{(148 \text{ k} - \text{ft.} + 332 \text{ k} - \text{ft.} + 1505 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

$= 2.430$ ksi (comp.) $\leq$ 4.200 ksi Ok

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \qquad \text{(b)}$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} - \frac{(975 \text{ kips})(30.92 \text{ in.})}{15421 \text{ in.}^3} + \frac{(3168 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ \frac{(148 \text{ k} - \text{ft.} + 332 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

$= 1.938$ ksi (comp.) $\leq$ 3.150 ksi Ok

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} + \frac{(975 \text{ kips})(30.92 \text{ in.})}{14915 \text{ in.}^3} - \frac{(3168 \text{ k-ft.} + 0)(12 \text{ in./ft.})}{14915 \text{ in.}^3}$$

$$- \frac{(148 \text{ k-ft.} + 332 \text{ k-ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3} - 0.8\frac{(1505 \text{ k-ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3}$$

$= 0.346$ ksi (tension) ≤ 0.503 ksi Ok

Span 2:

Prestress Force after Losses:

$$F_s = A_{ps}(f_{pbt} - \Delta f_{pT}) = \left(3.366\text{ in.}^2\right)\left(201.96\text{ ksi} - 25.07\text{ ksi}\right) = 595\text{ kips}$$

Service Stresses:

@ 0.5 span 2:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t^{'}} \qquad \text{(a)}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} - \frac{(595\text{ kips})(30.24\text{ in.})}{15421\text{ in.}^3} + \frac{(1616\text{ k}-\text{ft.} + 0)(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$$+ \frac{(28\text{ k}-\text{ft.} + 62\text{ k}-\text{ft.} + 1085\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3}$$

= 1.250 ksi (comp.) ≤ 4.200 ksi Ok

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \qquad \text{(b)}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} - \frac{(595\text{ kips})(30.24\text{ in.})}{15421\text{ in.}^3} + \frac{(1616\text{ k}-\text{ft.} + 0)(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$$+ \frac{(28\text{ k}-\text{ft.} + 62\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3}$$

= 0.896 ksi (comp.) ≤ 3.150 ksi Ok

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} + \frac{(595\text{ kips})(30.24\text{ in.})}{14915\text{ in.}^3} - \frac{(1616\text{ k}-\text{ft.} + 0)(12\text{ in./ft.})}{14915\text{ in.}^3}$$

$$- \frac{(28\text{ k}-\text{ft.} + 62\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3} - 0.8\frac{(1085\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3}$$

= 0.062 ksi (comp.) in compression Ok

@ 0.6 span 2:

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_t^{'}} \quad \text{(a)}$$

$$= \frac{595 \text{ kips}}{767 \text{ in.}^2} - \frac{(595 \text{ kips})(30.24 \text{ in.})}{15421 \text{ in.}^3} + \frac{(1552 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ \frac{(42 \text{ k} - \text{ft.} + 95 \text{ k} - \text{ft.} + 1095 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

= 1.219 ksi (comp.) ≤ 4.200 ksi Ok

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}} \quad \text{(b)}$$

$$= \frac{595 \text{ kips}}{767 \text{ in.}^2} - \frac{(595 \text{ kips})(30.24 \text{ in.})}{15421 \text{ in.}^3} + \frac{(1552 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ \frac{(42 \text{ k} - \text{ft.} + 95 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

= 0.861 ksi (comp.) ≤ 3.150 ksi Ok

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (0.8)\frac{M_{LL+IM}(12)}{S_b^{'}}$$

$$= \frac{595 \text{ kips}}{767 \text{ in.}^2} + \frac{(595 \text{ kips})(30.24 \text{ in.})}{14915 \text{ in.}^3} - \frac{(1552 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{14915 \text{ in.}^3}$$

$$- \frac{(42 \text{ k} - \text{ft.} + 95 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3} - 0.8\frac{(1095 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3}$$

= 0.078 ksi (comp.) in compression Ok

Check Fatigue Stresses after Losses

Span 1:

Determine if section is cracked for fatigue investigions:

@ 0.4 span 1:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (1.5)\frac{M_{FL+IM}(12)}{S_b^{'}}$$

$$= \frac{975\text{ kips}}{767\text{ in.}^2} + \frac{(975\text{ kips})(30.92\text{ in.})}{14915\text{ in.}^3} - \frac{(3041\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{14915\text{ in.}^3}$$

$$- \frac{(157\text{ k}-\text{ft.}+354\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3} - 1.5\frac{(478\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3}$$

$= 0.051$ ksi (comp.) $\leq$ 0.251 ksi Use uncracked section properties

@ 0.5 span 1:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1}+M_{DW1})(12)}{S_b} - \frac{(M_{DC2}+M_{DW2})(12)}{S_b'} - (1.5)\frac{M_{FL+IM}(12)}{S_b'}$$

$$= \frac{975\text{ kips}}{767\text{ in.}^2} + \frac{(975\text{ kips})(30.92\text{ in.})}{14915\text{ in.}^3} - \frac{(3168\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{14915\text{ in.}^3}$$

$$- \frac{(148\text{ k}-\text{ft.}+332\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3} - 1.5\frac{(462\text{ k}-\text{ft.})(12\text{ in./ft.})}{18550\text{ in.}^3}$$

$= 0.015$ ksi (tension) $\leq$ 0.251 ksi Use uncracked section properties

Fatigue Stresses:

@ 0.4 span 1:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1}+M_{DW1})(12)}{S_t} + \frac{(M_{DC2}+M_{DW2})(12)}{S_t'}\right] + 1.5\frac{M_{FL+IM}(12)}{S_t'}$$

$$= 0.5\left(\frac{975\text{ kips}}{767\text{ in.}^2}\right) - 0.5\frac{(975\text{ kips})(30.92\text{ in.})}{15421\text{ in.}^3} + 0.5\frac{(3041\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$$+ 0.5\frac{(157\text{ k}-\text{ft.}+354\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3} + 1.5\frac{(478\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3}$$

$= 1.159$ ksi (comp.) $\leq$ 2.800 ksi Ok

@ 0.5 span 1:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1}+M_{DW1})(12)}{S_t} + \frac{(M_{DC2}+M_{DW2})(12)}{S_t'}\right] + 1.5\frac{M_{FL+IM}(12)}{S_t'}$$

$$= 0.5\left(\frac{975\text{ kips}}{767\text{ in.}^2}\right) - 0.5\frac{(975\text{ kips})(30.92\text{ in.})}{15421\text{ in.}^3} + 0.5\frac{(3168\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{15421\text{ in.}^3}$$

$$+ 0.5\frac{(148\text{ k}-\text{ft.}+332\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3} + 1.5\frac{(462\text{ k}-\text{ft.})(12\text{ in./ft.})}{36755\text{ in.}^3}$$

$= 1.195$ ksi (comp.) $\leq$ 2.800 ksi Ok

Span 2:

Determine if section is cracked for fatigue investigions:

@ 0.5 span 2:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (1.5)\frac{M_{FL+IM}(12)}{S_b^{'}}$$

$$= \frac{595 \text{ kips}}{767 \text{ in.}^2} + \frac{(595 \text{ kips})(30.24 \text{ in.})}{14915 \text{ in.}^3} - \frac{(1616 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{14915 \text{ in.}^3}$$

$$- \frac{(28 \text{ k} - \text{ft.} + 62 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3} - 1.5\frac{(348 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3}$$

$= 0.286$ ksi (comp.) $\leq$ 0.251 ksi Use uncracked section properties

@ 0.6 span 2:

$$f_b = \frac{F_s}{A} + \frac{F_s e}{S_b} - \frac{(M_{DC1} + M_{DW1})(12)}{S_b} - \frac{(M_{DC2} + M_{DW2})(12)}{S_b^{'}} - (1.5)\frac{M_{FL+IM}(12)}{S_b^{'}}$$

$$= \frac{595 \text{ kips}}{767 \text{ in.}^2} + \frac{(595 \text{ kips})(30.24 \text{ in.})}{14915 \text{ in.}^3} - \frac{(1552 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{14915 \text{ in.}^3}$$

$$- \frac{(42 \text{ k} - \text{ft.} + 95 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3} - 1.5\frac{(359 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{18550 \text{ in.}^3}$$

$= 0.296$ ksi (comp.) $\leq$ 0.251 ksi Use uncracked section properties

Fatigue Stresses:

@ 0.5 span 2:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_t^{'}}$$

$$= 0.5\left(\frac{595 \text{ kips}}{767 \text{ in.}^2}\right) - 0.5\frac{(595 \text{ kips})(30.24 \text{ in.})}{15421 \text{ in.}^3} + 0.5\frac{(1616 \text{ k} - \text{ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ 0.5\frac{(28 \text{ k} - \text{ft.} + 62 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3} + 1.5\frac{(348 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

$= 0.618$ ksi (comp.) $\leq$ 2.800 ksi Ok

@ 0.6 span 2:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})(12)}{S_t} + \frac{(M_{DC2} + M_{DW2})(12)}{S_t^{'}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_t^{'}}$$

$$= 0.5\left(\frac{595 \text{ kips}}{767 \text{ in.}^2}\right) - 0.5\frac{(595 \text{ kips})(30.24 \text{ in.})}{15421 \text{ in.}^3} + 0.5\frac{(1552 \text{ k-ft.} + 0)(12 \text{ in./ft.})}{15421 \text{ in.}^3}$$

$$+ 0.5\frac{(42 \text{ k-ft.} + 95 \text{ k-ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3} + 1.5\frac{(359 \text{ k-ft.})(12 \text{ in./ft.})}{36755 \text{ in.}^3}$$

= 0.607 ksi (comp.) ≤ 2.800 ksi Ok

<u>Check Factored Flexural Resistance</u>

Span 1:

Strength I Moment:

@ 0.4 Span 1

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

= 1.25(3041 k-ft. + 157 k-ft.) + 1.5(0 + 354 k-ft.) + 1.75(1548 k-ft.)

= 7238 k-ft.

@ 0.5 Span 1

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

= 1.25(3168 k-ft. + 148 k-ft.) + 1.5(0 + 332 k-ft.) + 1.75(1505 k-ft.)

= 7277 k-ft. governs

Factored Flexural Resistance:

$M_r = \phi M_n$

Calculate Compression Block Depth (assume rectangular):

$$c = \frac{A_{ps} f_{pu}}{0.85 f_c^{'} \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$ (Eq. 5.7.3.1.1-4)

In which:

f'_c = deck concrete strength = 3.5 ksi

$\beta_1 = 0.65 \le 0.85 - 0.05(f_c^{'} - 4.0) \le 0.85$ (5.7.2.2)

$= 0.65 \le 0.85 - 0.05(3.5 \text{ ksi} - 4.0) \le 0.85$

$= \quad 0.65 \leq 0.875 \leq 0.85$

$= \quad 0.85$

b = effective flange width = 54 in.

k = 0.28

d_p = deck thickness + C_t + e (center)

= 8 in. + 35.40 in. + 30.92 in.

= 74.32 in.

$$c = \frac{(5.814 \text{ in.}^2)(270 \text{ ksi})}{0.85(3.5 \text{ ksi})(0.85)(54 \text{ in.}) + 0.28(5.814 \text{ in.}^2)\dfrac{270 \text{ ksi}}{74.32 \text{ in.}}}$$

= 11.02 in.

$a = \beta_1 c$

= 0.85(11.02 in.)

= 9.37 in. ≥ 8 in. Therefore Flanged Section

Calculate Compression Block Depth (flanged section):

$$c = \frac{A_{ps} f_{pu} - 0.85 f_c^{'} (b - b_w) h_f}{0.85 f_c^{'} \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.7.3.1.1-3)}$$

$$= \frac{(5.814 \text{ in.}^2)(270 \text{ ksi}) - 0.85(3.5 \text{ ksi})(54 \text{ in.} - 42 \text{ in.})(8 \text{ in.})}{0.85(3.5 \text{ ksi})(0.85)(42 \text{ in.}) + 0.28(5.814 \text{ in.}^2)\dfrac{270 \text{ ksi}}{74.32 \text{ in.}}}$$

= 11.45 in.

$a = \beta_1 c$

= 0.85(11.45 in.)

= 9.73 in.

Calculate Nominal Flexural Resistance (flanged section):

$$M_n = \left[A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) + 0.85 f_c^{'} (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right) \right] \left(\frac{1}{12} \right) \qquad \text{(Eq. 5.7.3.2.2-1)}$$

In which:

Check $f_{pe} \geq 0.5 f_{pu}$ to verify use of Eq. 5.7.3.1.1-1 (5.7.3.1.1)

$$f_{pe} = f_{pu} - \Delta f_{pT}$$
$$= 270 \text{ ksi} - 34.28 \text{ ksi}$$
$$= 235.72 \text{ ksi} \geq 0.5(270 \text{ ksi}) = 135 \text{ ksi} \quad \text{Ok}$$

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(Eq. 5.7.3.1.1-1)}$$
$$= 270 \text{ ksi}\left(1 - 0.28\frac{11.45 \text{ in.}}{74.32 \text{ in.}}\right)$$
$$= 258 \text{ ksi}$$

$$M_n = \left[(5.814 \text{ in.}^2)(258 \text{ ksi})\left(74.32 \text{ in.} - \frac{9.73 \text{ in.}}{2}\right)\right]\left(\frac{1}{12 \text{ in./ft.}}\right)$$
$$+ \left[0.85(3.5 \text{ ksi})(54 \text{ in.} - 42 \text{ in.})(8 \text{ in.})\left(\frac{9.73 \text{ in.}}{2} - \frac{8 \text{ in.}}{2}\right)\right]\left(\frac{1}{12 \text{ in./ft.}}\right)$$
$$= 8703 \text{ k-ft.}$$

Calculate ϕ:

$$\phi = 0.75 \leq 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \leq 1.0 \quad \text{(Eq. 5.5.4.2.1-1)}$$

In which:

d_t = deck thickness + depth of beam − distance from bottom of beam to bottom row of strands

= 8 in. + 72 in. − 2 in.

= 78 in.

$$\phi = 0.75 \leq 0.583 + 0.25\left(\frac{78 \text{ in.}}{9.73 \text{ in.}} - 1\right) \leq 1.0$$
$$= 0.75 \leq 2.34 \leq 1.0$$
$$= 1.0$$

$$M_r = 1.0(8703 \text{ k-ft.})$$

= 8703 k-ft. ≥ 7277 k-ft. Ok

Check Minimum Prestressing Steel:

$M_r \geq M_{cr}$ (5.7.3.3.2)

In which:

$f_r = 0.24\sqrt{f_c^{'}}$ (5.4.2.6)

$= 0.24\sqrt{7.0\,\text{ksi}}$

= 0.63 ksi

$$f_{cpe} = \frac{F_s}{A} + \frac{F_s e}{S_b}$$

$$= \frac{975\,\text{kips}}{767\,\text{in.}^2} + \frac{(975\,\text{kips})(30.92\,\text{in.})}{14915\,\text{in.}^3}$$

= 3.29 ksi

M_{DC1} = 3041 k-ft. (use the smaller value at 0.4 and 0.5)

$$M_{cr} = \gamma_3\left[\frac{S_b^{'}\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)}{12} - M_{DC1}\left(\frac{S_b^{'}}{S_b} - 1\right)\right] \quad \text{(Eq. 5.7.3.3.2-1)}$$

$$= 1.00\left[\frac{\left(18550\,\text{in.}^3\right)\left(1.6(0.63\,\text{ksi}) + 1.1(3.29\,\text{ksi})\right)}{(12\,\text{in./ft.})} - 3041\,\text{k-ft.}\left(\frac{18550\,\text{in.}^3}{14915\,\text{in.}^3} - 1\right)\right]$$

= 6411 k-ft.

M_r ≥ 6411 k-ft.

8703 k-ft. ≥ 6411 k-ft. Ok

Span 2:

Strength I Moment:

@ 0.5 Span 2

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

= 1.25(1616 k-ft. + 28 k-ft.) + 1.5(0 + 62 k-ft.) + 1.75(1085 k-ft.)

= 4047 k-ft.

@ 0.6 Span 2

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

= 1.25(1552 k-ft. + 42 k-ft.) + 1.5(0 + 95 k-ft.) + 1.75(1095 k-ft.)

= 4051 k-ft. governs

Factored Flexural Resistance:

$M_r = \phi M_n$

Calculate Compression Block Depth (assume rectangular):

$$c = \frac{A_{ps}f_{pu}}{0.85f'_c\beta_1 b + kA_{ps}\frac{f_{pu}}{d_p}}$$ (Eq. 5.7.3.1.1-4)

In which:

f'_c = deck concrete strength = 3.5 ksi

$\beta_1 = 0.65 \le 0.85 - 0.05(f'_c - 4.0) \le 0.85$ (5.7.2.2)

$= 0.65 \le 0.85 - 0.05(3.5\text{ ksi} - 4.0) \le 0.85$

$= 0.65 \le 0.875 \le 0.85$

= 0.85

b = effective flange width = 54 in.

k = 0.28

d_p = deck thickness + C_t + e (center)

= 8 in. + 35.40 in. + 30.24 in.

= 73.64 in.

$$c = \frac{(3.366\text{ in.}^2)(270\text{ ksi})}{0.85(3.5\text{ ksi})(0.85)(54\text{ in.}) + 0.28(3.366\text{ in.}^2)\frac{270\text{ ksi}}{73.64\text{ in.}}}$$

$= 6.49$ in.

$a = \beta_1 c$

$= 0.85(6.49$ in.)

$= 5.52$ in. $\leq$ 8 in. Therefore Rectangular Section

Calculate Nominal Flexural Resistance:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)\left(\frac{1}{12}\right) \quad \text{(Eq. 5.7.3.2.2-1)}$$

In which:

Check $f_{pe} \geq 0.5\, f_{pu}$ to verify use of Eq. 5.7.3.1.1-1 (5.7.3.1.1)

$f_{pe} = f_{pu} - \Delta f_{pT}$

$= 270$ ksi $-$ 25.07 ksi

$= 244.93$ ksi $\geq$ 0.5(270 ksi) = 135 ksi Ok

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(Eq. 5.7.3.1.1-1)}$$

$$= 270\text{ ksi}\left(1 - 0.28\frac{6.49\text{ in.}}{73.64\text{ in.}}\right)$$

$= 263$ ksi

$$M_n = (3.366\text{ in.}^2)(263\text{ ksi})\left(73.64\text{ in.} - \frac{5.52\text{ in.}}{2}\right)\left(\frac{1}{12\text{ in./ft.}}\right)$$

$= 5229$ k-ft.

Calculate ϕ:

$$\phi = 0.75 \leq 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \leq 1.0 \quad \text{(Eq. 5.5.4.2.1-1)}$$

In which:

d_t = deck thickness + depth of beam − distance from bottom of beam to bottom row of strands

= 8 in. + 72 in. − 2 in.

= 78 in.

$$\phi = 0.75 \le 0.583 + 0.25\left(\frac{78\text{ in.}}{6.49\text{ in.}} - 1\right) \le 1.0$$

$$= 0.75 \le 3.34 \le 1.0$$

= 1.0

M_r = 1.0(5229 k-ft.)

= 5229 k-ft. ≥ 4051 k-ft. Ok

Check Minimum Prestressing Steel:

$M_r \ge M_{cr}$ (5.7.3.3.2)

In which:

$$f_r = 0.24\sqrt{f'_c} \quad (5.4.2.6)$$

$$= 0.24\sqrt{7.0\text{ ksi}}$$

= 0.63 ksi

$$f_{cpe} = \frac{F_s}{A} + \frac{F_s e}{S_b}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} + \frac{(595\text{ kips})(30.24\text{ in.})}{14915\text{ in.}^3}$$

= 1.98 ksi

M_{DC1} = 1552 k-ft. (use the smaller value at 0.5 and 0.6)

$$M_{cr} = \gamma_3\left[\frac{S'_b(\gamma_1 f_r + \gamma_2 f_{cpe})}{12} - M_{DC1}\left(\frac{S'_b}{S_b} - 1\right)\right] \quad \text{(Eq. 5.7.3.3.2-1)}$$

$$= 1.0\left[\frac{\left(18550\text{ in.}^3\right)\left(1.6(0.63\text{ ksi}) + 1.1(1.98\text{ ksi})\right)}{\left(12\text{ in./ft.}\right)} - 1552\text{ k-ft.}\left(\frac{18550\text{ in.}^3}{14915\text{ in.}^3} - 1\right)\right]$$

$= 4547$ k-ft.

$M_r \geq 4547$ k-ft.

5229 k-ft. ≥ 4547 k-ft. Ok

Design Negative Moment Region

Strength I Moment

$M_u = 1.25(M_{DC1} + M_{DC2}) + 1.5(M_{DW1} + M_{DW2}) + 1.75(M_{LL+IM})$

$= 1.25(0 + -195) + 1.5(0 + -439) + 1.75(-1420)$

$= -3387$ k-ft.

Estimate Negative Moment Reinforcement

$$R_n = \frac{M_u(12)}{\phi b(d_s)^2}$$

In which:

$$d_s \approx \text{Beam Depth} + \frac{\text{deck thickness}}{2} = 72\text{ in.} + \frac{8\text{ in.}}{2} = 76\text{ in.}$$

$$R_n = \frac{(3387\text{ k-ft.})(12\text{ in./ft.})}{0.9(26\text{ in.})(76\text{ in.})^2}$$

$= 0.301$ ksi

$$\rho = \frac{0.85f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right]$$

$$= \frac{0.85(7.0\text{ ksi})}{60\text{ ksi}}\left[1 - \sqrt{1 - \frac{2(0.301\text{ ksi})}{0.85(7.0\text{ ksi})}}\right]$$

$= 0.00515$

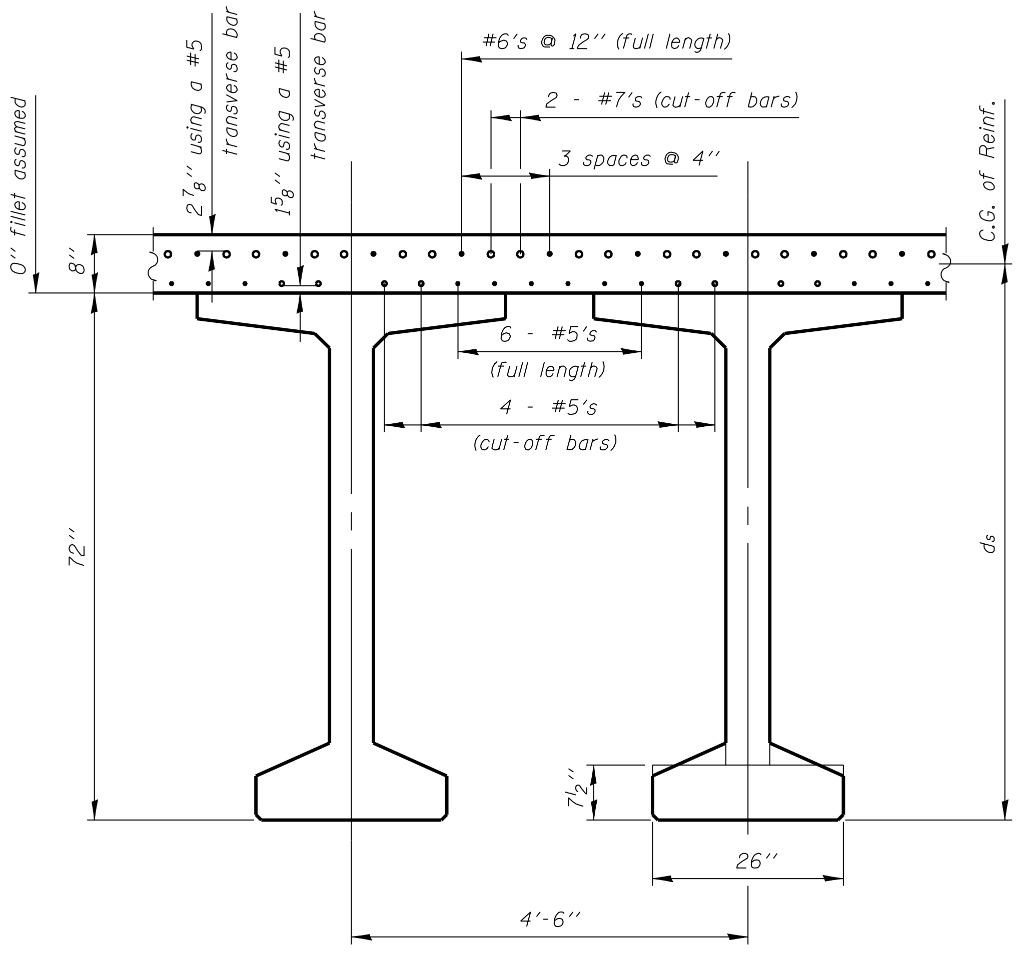
Estimated total reinforcement at pier:

$A_s = \rho bd = 0.00515(26\text{ in.})(76\text{ in.}) = 10.18\text{ in.}^2$

Estimated continuing reinforcement at cutoff point:

$$A_s = \frac{A_s(\text{total})}{3} = \frac{10.18 \text{ in.}^2}{3} = 3.39 \text{ in.}^2$$

Check Trial Reinforcement Configuration Below



CROSS SECTION AT PIER

Calculate Area of Reinforcement:

Top (full length)	=	#6 @ 12"	=	(0.44 in.^{2})(4.5)	=	1.98 in.^{2}
Top (cut-off)	=	2 - #7 @12"	=	2(0.60 in.^{2})(4.5)	=	5.40 in.^{2}
Bottom (full length)	=	6 - #5	=	6(0.31 in.^{2})	=	1.86 in.^{2}
Bottom (cut-off)	=	4 - #5	=	4(0.31 in.^{2})	=	1.24 in.^{2}

Totals:

A_s (full length) $= 1.98 \text{ in.}^2 + 1.86 \text{ in.}^2$

$= 3.84 \text{ in.}^2$

A_s (total) $= 1.98 \text{ in.}^2 + 5.40 \text{ in.}^2 + 1.86 \text{ in.}^2 + 1.24 \text{ in.}^2$

$= 10.48 \text{ in.}^2 \geq 10.18 \text{ in.}^2$ ok

Check continuing reinforcement (minimum 1/3 reinforcement to continue past cut-off):

$$\frac{A_s(\text{full length})}{A_s(\text{total})} \geq 0.333$$

$$\frac{3.84 \text{ in.}^2}{10.48 \text{ in.}^2} \geq 0.333$$

$0.366 \geq 0.333$ Ok

Calculate Center of Gravity of Reinforcement:

	A_s	d_s	$A_s d_s$
Top (full length)	1.98 in.^{2}	76.75 in.	151.97 in.^{3}
Top (cut-off)	5.40 in.^{2}	76.69 in.	414.13 in.^{3}
Bottom (full length)	1.86 in.^{2}	73.94 in.	137.53 in.^{3}
Bottom (cut-off)	1.24 in.^{2}	73.94 in.	91.69 in.^{3}
	10.48 in.^{2}		795.32 in.^{3}

$$d_s = \frac{795.32 \text{ in.}^3}{10.48 \text{ in.}^2} = 75.89 \text{ in.}$$

<u>Factored Flexural Resistance</u>

$M_r = \phi M_n$

Calculate Compression Block Depth (assume rectangular):

$$c = \frac{A_s f_s}{0.85 f'_c \beta_1 b} \qquad \text{(Eq. 5.7.3.1.1-4)}$$

In which:

f'_c = beam concrete strength = 7.0 ksi

$\beta_1 = 0.65 \leq 0.85 - 0.05(f'_c - 4.0) \leq 0.85$ (5.7.2.2)

$= 0.65 \leq 0.85 - 0.05(7.0 \text{ ksi} - 4.0) \leq 0.85$

$= 0.65 \leq 0.70 \leq 0.85$

$= 0.70$

$$f_s = f_y \quad \text{if } \frac{c}{d_s} \leq 0.6 \qquad (5.7.2.1)$$

$$c = \frac{(10.48 \text{ in.}^2)(60 \text{ ksi})}{0.85(7.0 \text{ ksi})(0.70)(26 \text{ in.})} = 5.81 \text{ in.}$$

$$\text{Check } \frac{c}{d_s} \leq 0.6 = \frac{5.81 \text{ in.}}{75.89 \text{ in.}} \leq 0.6 = 0.08 \leq 0.6 \qquad \text{Ok}$$

$a = \beta_1 c$

$= 0.70(5.81 \text{ in.})$

$= 4.07 \text{ in.} \leq 7.5$ Therefore Rectangular Section

Calculate Nominal Flexural Resistance (rectangular section):

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)\left(\frac{1}{12}\right) \qquad \text{(Eq. 5.7.3.2.2-1)}$$

$$= (10.48 \text{ in.}^2)(60 \text{ ksi})\left(75.89 \text{ in.} - \frac{4.07 \text{ in.}}{2}\right)\left(\frac{1}{12 \text{ in./ft.}}\right)$$

$= 3870$ k-ft.

Calculate ϕ:

$$\phi = 0.75 \leq 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \leq 0.9 \qquad \text{(Eq. 5.5.4.2.1-2)}$$

In which:

d_t = deck thickness + depth of beam − distance from top of slab to centroid of top row of longitudinal reinforcement bars

= 8 in. + 72 in. − 3.25 in.

= 76.75 in.

$$\phi = 0.75 \leq 0.65 + 0.15\left(\frac{76.75 \text{ in.}}{5.81 \text{ in.}} - 1\right) \leq 0.9 = 0.75 \leq 2.48 \leq 0.9 = 0.9$$

$M_r = 0.9(3870 \text{ k-ft.})$

$= 3483 \text{ k-ft.} \geq 3387 \text{ k-ft.}$ Ok

Check Minimum Reinforcement

$M_r \geq M_{cr}$ (5.7.3.3.2)

In which:

$f_r = 0.24\sqrt{f'_c}$ (5.4.2.6)

$= 0.24\sqrt{7.0\text{ ksi}}$

$= 0.63$ ksi

$$M_{cr} = \gamma_3\gamma_1 \frac{S'_{ts}(f_r)}{12}$$ (Eq. 5.7.3.3.2-1)

In which:

$$S'_{ts} = \frac{I'}{(y_t + \text{slab t})} = \frac{887639\text{ in.}^4}{(24.15\text{ in.} + 8\text{ in.})} = 27609\text{ in.}^3$$

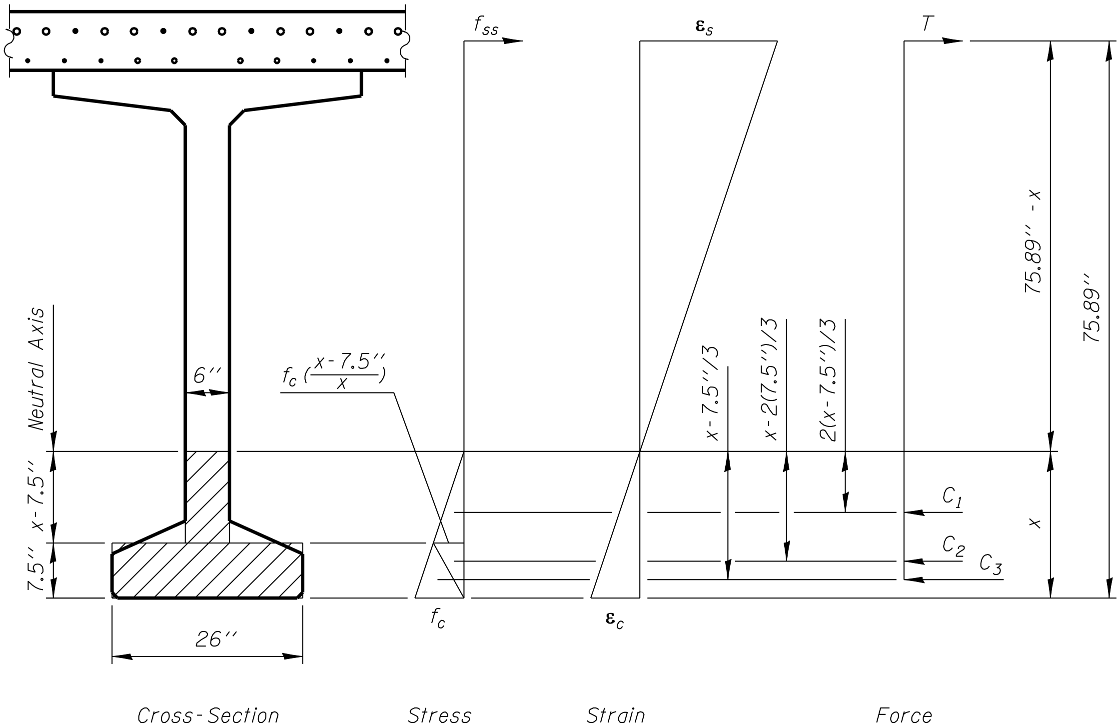
$$M_{cr} = (0.75)(1.6)\frac{(27609\text{ in.}^3)(0.63\text{ ksi})}{12\text{ in./ft.}}$$

$= 1739$ k-ft.

$M_r \geq 1739$ k-ft.

3483 k-ft. $\geq$ 1739 k-ft. Ok

Calculation of Stresses for Service and Fatigue Limit States



STRESS STRAIN DIAGRAM AT PIER

Determine expression for f_{ss}:

$$f_{ss} = f_c n\left(\frac{d_s - x}{x}\right)$$

$$= f_c(6)\left(\frac{75.89\text{ in.} - 19.63\text{ in.}}{19.63\text{ in.}}\right)$$

$$= 17.20 f_c$$

Determine x:

$$A_s n(d_s - x) = (x - h_f)\left(\frac{x - h_f}{2}\right)b_w + (x - h_f)\left(\frac{h_f}{2}\right)b + \left(\frac{h_f}{2}\right)bx$$

Insert known variables and put into quadratic form (units not shown for clarity):

$$10.48(6)(75.89 - x) = (x - 7.5)\left(\frac{x - 7.5}{2}\right)6 + (x - 7.5)\left(\frac{7.5}{2}\right)26 + \left(\frac{7.5}{2}\right)26x$$

$$4771.96 - 62.88x = 3x^2 - 22.5x - 22.5x + 168.75 + 97.5x - 731.25 + 97.5x$$

$$0 = 3x^2 + 212.88x - 5334.46$$

Solve for x:

$$x = 19.63 \text{ in.}$$

Determine S_{bc}:

$$S_{bc} = A_s n\left(\frac{d_s - x}{x}\right)(d_s - x) + \left(\frac{x - h_f}{x}\right)\left(\frac{x - h_f}{2}\right)b_w\left(\frac{2(x - h_f)}{3}\right)$$

$$+ \left(\frac{x - h_f}{x}\right)\left(\frac{h_f}{2}\right)b\left(x - \frac{2h_f}{3}\right) + \left(\frac{h_f}{2}\right)b\left(x - \frac{h_f}{3}\right)$$

$$= \left(10.48 \text{ in.}^2\right)(6 \text{ in.})\left(\frac{75.89 \text{ in.} - 19.63 \text{ in.}}{19.63 \text{ in.}}\right)(75.89 \text{ in.} - 19.63 \text{ in.})$$

$$+ \left(\frac{19.63 \text{ in.} - 7.5 \text{ in.}}{19.63 \text{ in.}}\right)\left(\frac{19.63 \text{ in.} - 7.5 \text{ in.}}{2}\right)(6 \text{ in.})\left(\frac{2(19.63 \text{ in.} - 7.5 \text{ in.})}{3}\right)$$

$$+ \left(\frac{19.63 \text{ in.} - 7.5 \text{ in.}}{19.63 \text{ in.}}\right)\left(\frac{7.5 \text{ in.}}{2}\right)(26 \text{ in.})\left(19.63 \text{ in.} - \frac{2(7.5 \text{ in.})}{3}\right)$$

$$+ \left(\frac{7.5 \text{ in.}}{2}\right)(26 \text{ in.})\left(19.63 \text{ in.} - \frac{7.5 \text{ in.}}{3}\right)$$

$$= 12872 \text{ in.}^3$$

Check Control of Cracking by Distribution of Reinforcement

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.7.3.4-1)}$$

In which:

$$d_c = 2.25 \text{ in.} + 0.625 \text{ in.} + 0.5(0.75 \text{ in.}) = 3.25 \text{ in.}$$

$$h = 72 \text{ in.} + 8 \text{ in.} = 80 \text{ in.}$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

$$= 1 + \frac{3.25 \text{ in.}}{0.7(80 \text{ in.} - 3.25 \text{ in.})}$$

$$= 1.06$$

γ_e = 0.75 for Class 2 exposure

$$f_c = \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_{bc}}$$

$$= \frac{(195\ k-ft. + 439\ k-ft. + 1420\ k-ft.)(12\ in./ft.)}{12872\ in.^3}$$

$$= 1.915\ ksi$$

$$f_{ss} = 17.20 f_c$$

$$= 17.20(1.915\ ksi)$$

$$= 32.94\ ksi$$

$$s \le \frac{700(0.75)}{1.06(32.94\ ksi)} - 2(3.25\ in.)$$

4 in. ≤ 8.5 in. Ok

Check Fatigue in Reinforcement

$\gamma(\Delta f) \le (\Delta F)_{TH}$ (Eq. 5.5.3.1-1)

In which:

M_{FL+IM} (fatigue truck range) = 0 to -339 k-ft. (from computer software)

Calculate $(\Delta F)_{TH}$:

$(\Delta F)_{TH} = 24 - 0.33 f_{min}$ (Eq. 5.5.3.2-1)

In which:

$$f_c = \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}} + 1.5\frac{(M_{FL+IM(min)})(12)}{S_{bc}}$$

$$= \frac{(195\ k-ft. + 439\ k-ft.)(12\ in./ft.)}{12872\ in.^3} + 1.5\frac{(0)(12\ in./ft.)}{12872\ in.^3}$$

$$= 0.591\ ksi$$

$$f_{min} = f_{ss} = 17.20 f_c$$

$$= 17.20(0.591\ ksi)$$

$$= 10.17\ ksi$$

$$(\Delta F)_{TH} = 24 - 0.33(10.17\ ksi) = 20.64\ ksi$$

Calculate $\gamma(\Delta f)$:

$\gamma = 1.5$ (for Fatigue I load combination)

$$f_c = \frac{(M_{FL+IM(\text{fatigue truck range})})(12)}{S_{bc}}$$

$$= \frac{(339 \text{ k} - \text{ft.})(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$= 0.316$ ksi

$\Delta f = f_{ss} = 17.20 f_c$

$= 17.20(0.316 \text{ ksi})$

$= 5.44$ ksi

$\gamma(\Delta f) = 1.5(5.44 \text{ ksi})$

$= 8.16$ ksi

$\gamma(\Delta f) \leq (\Delta F)_{TH}$

8.16 ksi $\leq$ 20.64 ksi Ok

<u>Check Compressive Service and Fatigue Stresses Near Pier</u>

The distance from centerline of pier to transfer point equals the distance from the centerline of pier to the end of the beam plus the transfer length.

Distance from centerline of pier to transfer point for spans 1 and 2:

$$\frac{6 \text{ in.}}{12 \text{ in./ft.}} + 2.5 \text{ ft.} = 3 \text{ ft.}$$

@ 0.979 span 1:

Moments (from computer software):

M_{DC1} = 266 k-ft.

M_{DW1} = 0 k-ft.

M_{DC2} = -170 k-ft.

M_{DW2} = -383 k-ft.

M_{LL+IM} = -1081 k-ft.

M_{FL+IM} = -293 k-ft.

Service Stresses:

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_{bc}} \qquad (a)$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} + \frac{(975 \text{ kips})(9.61 \text{ in.})}{12872 \text{ in.}^3} - \frac{(266 \text{ k-ft.} + 0)(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$$+ \frac{(170 \text{ k-ft.} + 383 \text{ k-ft.} + 1081 \text{ k-ft.})(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$= 3.274$ ksi (comp.) $\leq$ 4.200 ksi Ok

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}} \qquad (b)$$

$$= \frac{975 \text{ kips}}{767 \text{ in.}^2} + \frac{(975 \text{ kips})(9.61 \text{ in.})}{12872 \text{ in.}^3} - \frac{(266 \text{ k-ft.} + 0)(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$$+ \frac{(170 \text{ k-ft.} + 383 \text{ k-ft.})(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$= 2.267$ ksi (comp.) $\leq$ 3.150 ksi Ok

Fatigue Stresses:

$$f_b = 0.5\left[\frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_{bc}}$$

$$= 0.5\left(\frac{975 \text{ kips}}{767 \text{ in.}^2}\right) + 0.5\frac{(975 \text{ kips})(9.61 \text{ in.})}{12872 \text{ in.}^3} - 0.5\frac{(266 \text{ k-ft.} + 0)(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$$+ 0.5\frac{(170 \text{ k-ft.} + 383 \text{ k-ft.})(12 \text{ in./ft.})}{12872 \text{ in.}^3} + 1.5\frac{(293 \text{ k-ft.})(12 \text{ in./ft.})}{12872 \text{ in.}^3}$$

$= 1.543$ ksi (comp.) $\leq$ 2.800 ksi Ok

@ 0.03 span 2:

Moments (from computer software):

M_{DC1} = 188 k-ft.

M_{DW1} = 0 k-ft.

M_{DC2} = -175 k-ft.

M_{DW2} = -393 k-ft.

M_{LL+IM} = -1163 k-ft.

M_{FL+IM} = -328 k-ft.

Service Stresses:

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2} + M_{LL+IM})(12)}{S_{bc}} \quad \text{(a)}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} + \frac{(595\text{ kips})(30.24\text{ in.})}{12872\text{ in.}^3} - \frac{(188\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{12872\text{ in.}^3}$$

$$+ \frac{(175\text{ k}-\text{ft.}+393\text{ k}-\text{ft.}+1163\text{ k}-\text{ft.})(12\text{ in./ft.})}{12872\text{ in.}^3}$$

= 3.612 ksi (comp.) ≤ 4.200 ksi Ok

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}} \quad \text{(b)}$$

$$= \frac{595\text{ kips}}{767\text{ in.}^2} + \frac{(595\text{ kips})(30.24\text{ in.})}{12872\text{ in.}^3} - \frac{(188\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{12872\text{ in.}^3}$$

$$+ \frac{(175\text{ k}-\text{ft.}+393\text{ k}-\text{ft.})(12\text{ in./ft.})}{12872\text{ in.}^3}$$

= 2.528 ksi (comp.) ≤ 3.150 ksi Ok

Fatigue Stresses:

$$f_b = 0.5\left[\frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})(12)}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})(12)}{S_{bc}}\right] + 1.5\frac{M_{FL+IM}(12)}{S_{bc}}$$

$$= 0.5\left(\frac{595\text{ kips}}{767\text{ in.}^2}\right) + 0.5\frac{(595\text{ kips})(30.24\text{ in.})}{12872\text{ in.}^3} - 0.5\frac{(188\text{ k}-\text{ft.}+0)(12\text{ in./ft.})}{12872\text{ in.}^3}$$

$$+ 0.5\frac{(175\text{ k}-\text{ft.}+393\text{ k}-\text{ft.})(12\text{ in./ft.})}{12872\text{ in.}^3} + 1.5\frac{(328\text{ k}-\text{ft.})(12\text{ in./ft.})}{12872\text{ in.}^3}$$

= 1.723 ksi (comp.) ≤ 2.800 ksi Ok

Check Cutoff Points

Calculate capacity of continuing reinforcement using the same procedure given above. Next use this capacity to determine the cutoff location by comparing to the applied moments in a moment envelope. Then check distribution and fatigue using the same procedure given above. Finally calculate the embedment length of the cutoff reinforcement using the provisions of Art. 5.11.1.2.3. The cutoff points in each span will have to be determined and checked since the spans are not symmetrical.

Calculate Camber and Deflection

See example 1 for sample calculations.

Shear Design *(5.8)*

As showing full calculations in this design guide for every location along the beam is lengthy and unnecessary, full shear calculations will be shown for the critical section near the pier for span 1, and only tabulated results will be shown for the rest of the sections along the beam.

Location of Critical Section *(5.8.3.2)*

Taking the location of the critical section for shear at 0.72h from the face of the support at the pier gives a location of:

x_{Crit} = 0.72h + 1.25 ft. (face of support to CL bearing)

$= 0.72(72\text{ in. beam} + 8\text{ in. slab})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) + 1.25\text{ ft.}$

= 6.05 ft. from CL bearing of pier

The calculated values of the maximum and minimum Strength I shears and moments, factored, distributed, and including impact, at this location have been found to be:

M_u^+ = 405.9 k-ft. (from computer software)

M_u^- = -1785.7 k-ft. (from computer software)

V_u^+ = -91.9 k-ft. (from computer software)

V_u^- = -276 k-ft. (from computer software)

Nominal Shear Resistance *(5.8.3.3)*

The maximum permitted spacing based upon nominal shear resistance is taken as:

$$s \leq \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$

Where the required variables are as calculated below.

Effective Shear Depth d_v *(5.8.2.9)*

The effective shear depth, d_v, is taken as:

$$d_v = d_e - \frac{a}{2} \quad \text{(C5.8.2.9)}$$

Where:

d_e = 75.89 in. (see d_s in moment calculations)

a = 4.07 in. (see moment calculations)

$$d_v = 75.89 \text{ in.} - \left(\frac{4.07 \text{ in.}}{2}\right) \quad \text{(C5.8.2.9)}$$

$$= 73.86 \text{ in.}$$

d_v need not be taken as less than the greater of $0.9d_e$ and 0.72h.

$0.9d_e$ = 0.9(75.89 in.)

= 68.30 in.

0.72h = 0.72(80 in.)

= 57.6 in.

Since d_v is greater than both $0.9d_e$ and 0.72h, it is the controlling value.

d_v = 73.86 in.

Vertical Component of Prestressing Force V_p

$V_p = A_{ps}^{harped} f_{px} \sin\Psi$

Determine Transfer Lengths, Development Lengths, and f_{px} (5.11.4)

Transfer Length $= 60d_b - x_{brg}$ (5.11.4.1)

$= 60(0.5 \text{ in.}) – 8 \text{ in.}$

$= 22 \text{ in.}$

The development length, l_d, may be found using the following equation:

$$l_d \geq K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$ (Eq. 5.11.4.2-1)

Where:

$K = 1.6$

$$f_{pe} = \left(\frac{100 - \%\text{ losses}}{100}\right)(f_{pbt})$$

$$= \left(\frac{100 - 17}{100}\right)(201.96 \text{ ksi})$$ (see moment calculations for losses)

$= 167.63 \text{ ksi}$

$d_b = 0.5 \text{ in.}$

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ (Eq. 5.7.3.1.1-1)

Where:

$f_{pu} = 270 \text{ ksi}$

$k = 0.28$ for low relaxation strands (Table 5.4.4.1-1)

$c = 11.24$ in. (see positive moment calculations)

$d_p = h – C_b + e_{End}$

$= 80 \text{ in.} – 36.60 \text{ in.} + 8.60 \text{ in.}$

$= 52 \text{ in.}$

$$f_{ps} = (270\text{ ksi})\left(1 - 0.28\frac{11.24\text{ in.}}{52\text{ in.}}\right) \quad \text{(Eq. 5.7.3.1.1-1)}$$

$$= 253.66\text{ ksi}$$

$$l_d \geq 1.6\left(253.66\text{ ksi} - \frac{2}{3}(167.63\text{ ksi})\right)(0.5\text{ in.}) \quad \text{(Eq. 5.11.4.2-1)}$$

$$\geq 113.5\text{ in.}$$

The critical section is at x = 6.05 ft. $\left(\frac{12\text{ in.}}{\text{ft.}}\right)$ = 72.6 in.

22 in. < 72.6 in. < 113.5 in. Therefore,

$$f_{px} = f_{pe} + \frac{l_{px} - l_{trans}}{l_d - l_{trans}}(f_{ps} - f_{pe}) \quad \text{(Eq. 5.11.4.2-3)}$$

Where:

f_{ps} = 253.66 ksi

f_{pe} = 167.63 ksi

l_{px} = 72.6 in.

l_d = 113.5 in.

l_{trans} = 22 in.

$$f_{px} = 167.63\text{ ksi} + \frac{72.6\text{ in.} - 22\text{ in.}}{113.5\text{ in.} - 22\text{ in.}}(253.66\text{ ksi} - 167.63\text{ ksi}) \quad \text{(Eq. 5.11.4.2-3)}$$

$$= 215.19\text{ ksi}$$

$$A_{ps}^{harped} = (A_{strand} * \#\text{ of harped strands})$$

$$= \left(\frac{0.153\text{ in.}^2}{\text{strand}}\right)(16\text{ harped strands})$$

$$= 2.448\text{ in.}^2$$

$$\Psi = \arctan\left(\frac{d_{harping}}{0.4L}\right)$$

Where:

$d_{harping}$ = h_{beam} – 3 in. – dist. from top strand to bottom of beam at CL span

$= 72 \text{ in.} - 3 \text{ in.} - 16 \text{ in.}$

$= 53 \text{ in.}$

$$L = 141.33 \text{ ft.} \left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 1696 \text{ in.}$$

$$\Psi = \arctan\left(\frac{53 \text{ in.}}{0.4(1696 \text{ in.})}\right)$$

$$= 4.47^\circ$$

$$V_p = (2.448 \text{ in.}^2)(215.9 \text{ ksi})(\sin 4.47^\circ)$$

$$= 41.06 \text{ k}$$

Shear Resistance Due to Concrete V_c

V_c = $0.0316\beta\sqrt{f'_c}\,b_v d_v$ (Eq. 5.8.3.3-3)

Where:

f'_c = 7 ksi

d_v = 73.86 in.

b_v = 6 in.

β = $\dfrac{4.8}{1+750\varepsilon_s}$ (assume Art. 5.8.2.5 satisfied, then check) (Eq. 5.8.3.4.2-1)

Where:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{\left(E_sA_s + E_pA_{ps}\right)} \geq 0$$ (Eq. 5.8.3.4.2-4)

Where:

N_u = 0 k

$|M_u|$ = $(1785.7 \text{ k-ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)$ = 21428.4 k-in.

Check that $|M_u| \geq |V_u - V_p|d_v$:

d_v = 73.86 k

V_u = 276.0 k

V_p = 41.06 k

$|V_u - V_p|d_v$ = $|276.0 \text{ k} - 41.06 \text{ k}|(73.86 \text{ in.})$

= 17351.5 k-in.

$|M_u| \geq |V_u - V_p|d_v$, use $|M_u|$.

A_{ps} = Area of prestressing steel on flexural tension side of member (in.^{2}). Determine how many strands are on flexural tension side of member:

<u>Strand Pattern at Critical Section (Harped Strands Only)</u>

By inspection, all of the harped strands are on the top half i.e. flexural tension side of the composite section.

To find the depth of a row of strands at the critical section for shear, d_{Row}^{Crit} :

$$d_{Row}^{Crit} = d_{Row}^{End} - (x_{Crit} + x_{brg}) \tan \Psi$$

$$d_{1T}^{Crit} = 69 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 62.7 \text{ in.}$$

$$d_{2T}^{Crit} = 67 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 60.7 \text{ in.}$$

$$d_{3T}^{Crit} = 65 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 58.7 \text{ in.}$$

$$d_{4T}^{Crit} = 63 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 56.7 \text{ in.}$$

$$d_{5T}^{Crit} = 61 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 54.7 \text{ in.}$$

$$d_{6T}^{Crit} = 59 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 52.7 \text{ in.}$$

$$d_{7T}^{Crit} = 57 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 50.7 \text{ in.}$$

$$d_{8T}^{Crit} = 55 \text{ in.} - (72.6 \text{ in.} + 8 \text{ in.}) \tan 4.51^\circ = 48.7 \text{ in.}$$

The depth to the flexural tension side of the member is found as follows:

0.5(72 in. beam + 8 in. slab) = 40 in. from bottom of beam

All of the harped strands are above this point.

$$A_{ps} = (A_{strand} * \text{\# of strands in flexural tension side of member})$$

$$= \left(\frac{0.153 \text{ in.}^2}{\text{strand}}\right)(16 \text{ strands in flexural tension side of member})$$

$$= 2.448 \text{ in.}^2$$

$$f_{po} = 0.7 f_{pu}$$

$= 0.7(270 \text{ ksi})$

$= 189 \text{ ksi}$

$E_s = 29000 \text{ ksi}$

$A_s = 10.48 \text{ in.}^2$ (see moment computations)

$E_p = 28500 \text{ ksi}$

$$\varepsilon_s = \frac{\left(\frac{|21428.4 \text{ k} - \text{in.}|}{73.86 \text{ in.}} + 0.5(0 \text{ k}) + |276 \text{ k} - 41.06 \text{ k}| - (2.448 \text{ in.}^2)(189 \text{ ksi})\right)}{(29000 \text{ ksi})(10.48 \text{ in.}^2) + (28500 \text{ ksi})(2.448 \text{ in.}^2)}$$

(Eq. 5.8.3.4.2-4)

$\varepsilon_s = 0.00017 \text{ in./in.}$

$$\beta = \frac{4.8}{1 + 750(0.00017 \text{ in./in.})}$$ (Eq. 5.8.3.4.2-1)

$= 4.26$

$$V_c = 0.0316(4.26)\sqrt{7 \text{ ksi}}(6 \text{ in.})(73.86 \text{ in.})$$ (Eq. 5.8.3.3-3)

$= 157.83 \text{ k}$

Required Spacing of Transverse Reinforcement for Nominal Shear Resistance

$$s \leq \frac{A_v f_y d_v \cot\theta}{\frac{V_u}{\phi} - V_p - V_c}$$

Where:

$$A_v = \left(\frac{0.20 \text{ in.}^2}{\text{leg}}\right)(2 \text{ legs of } \#4 \text{ stirrups})$$

$= 0.40 \text{ in.}^2$

$f_y = 60 \text{ ksi}$

d_v = 73.86 in.

θ = $29 + 3500\varepsilon_s$ (Eq. 5.8.3.4.2-3)

= 29 + 3500(0.00017 in./in.)

= 29.60°

V_u = 276.0 k

ϕ = 0.9

V_p = 41.06 k

V_c = 157.83 k

$$s \le \frac{(0.40\text{ in.}^2)(60\text{ ksi})(73.86\text{ in.})(\cot 29.60^\circ)}{\frac{276.0\text{ k}}{0.9} - 41.06\text{ k} - 157.83\text{ k}}$$

$\le$ 29.0 in.

Maximum Permitted Spacing of Transverse Reinforcement *(5.8.2.7)*

$$v_u = \frac{|V_u - \phi V_p|}{\phi\, b_v d_v}$$ (Eq. 5.8.2.9-1)

Where:

V_u = 276.0 k

V_p = 41.06 k

ϕ = 0.9

b_v = 6 in.

$$v_u = \frac{|276.0\text{ k} - (0.9)(41.06\text{ k})|}{(0.9)(6\text{ in.})(73.86\text{ in.})}$$

= 0.6 ksi

$0.125f'_c$ = 0.125(7 ksi)

= 0.875 ksi

$v_u < 0.125f'_c$, therefore:

s_{max} = $0.8d_v \leq 24.0$ in. (Eq. 5.8.2.7-2)

= 0.8(73.86 in.)

= 59.1 in. > 24 in.

The maximum spacing of shear reinforcement for strength at the critical section for shear is 24 in.

Minimum Transverse Reinforcement *(5.8.2.5)*

$$s \leq \frac{A_v f_y}{0.0316\sqrt{f'_c}\,b_v}$$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

f'_c = 7 ksi

b_v = 6 in.

$$s \leq \frac{(0.40 \text{ in.}^2)(60 \text{ ksi})}{0.0316\sqrt{7 \text{ ksi}}(6 \text{ in.})} = 47.8 \text{ in.}$$

Interface Shear Transfer Reinforcement *(5.8.4)*

The maximum spacing for interface shear reinforcement is:

$$s = \frac{12\mu A_{vf} f_y}{\frac{V_{ui}}{\phi} - cA_{cv}}$$

Where:

μ = 1.0

A_{vf} = 0.40 in.^{2}

f_y = 60 ksi

ϕ = 0.9

c = 0.28 ksi (5.8.4.3)

b_{vi} = top flange width = 42 in.

L_{vi} = 12 in.

$A_{cv} = b_{vi}L_{vi}$ (Eq. 5.8.4.1-6)

= (42 in.)(12 in.)

= 504 in.^{2}

$V_{ui} = v_{ui}A_{cv}$

Where:

$$v_{ui} = \frac{V_u}{b_{vi}d_v}$$ (Eq. 5.8.4.2-1)

Where:

V_u = 276.0 k

b_{vi} = top flange width = 42 in.

d_v = 73.86 in.

$$v_{ui} = \frac{276.0\text{ k}}{(42\text{ in.})(73.86\text{ in.})}$$ (Eq. 5.8.4.2-1)

= 0.09 ksi

V_{ui} = (0.09 ksi)(504 in.^{2})

= 45.4 k

Check that $\frac{V_{ui}}{\phi} \leq K_1 f'_c A_{cv}$

Where:

V_{ui} = 45.4 k

ϕ = 0.9

K_1 = 0.3 (5.8.4.3)

f'_c = 7 ksi

A_{cv} = 504 in.^{2} (Eq. 5.8.4.1-6)

$\frac{V_{ui}}{\phi} = \frac{45.4\ k}{0.9} = 50.4\ k$

$K_1 f'_c A_{cv}$ = (0.3)(7 ksi)(504 in.^{2}) = 1058.4 k > 50.4 k

Check that $\frac{V_{ui}}{\phi} \le K_2 A_{cv}$

Where:

V_{ui} = 45.4 k

ϕ = 0.9

K_2 = 1.8 ksi (5.8.4.3)

f'_c = 7 ksi

A_{cv} = 504 in.^{2} (Eq. 5.8.4.1-6)

$\frac{V_{ui}}{\phi} = \frac{45.4\ k}{0.9} = 50.4\ k$

$K_2 A_{cv}$ = (1.8 ksi)(504 in.^{2}) = 907.2 k > 50.4 k

For $\frac{V_{ui}}{\phi}$, use 50.4 k.

$$s = \frac{12(1.0)(0.4\ in.^2)(60\ ksi)}{50.4\ k - (0.28\ ksi)(504\ in.^2)} \quad \text{(Eq. i)}$$

$= -3.17$ in.

Since s < 0, no reinforcement is required for interface shear. Check the spacing required to satisfy $\frac{1.33V_{ui}}{\phi}$:

$$s = \frac{12(1.0)(0.4\ in.^2)(60\ ksi)}{1.33(50.4\ k) - (0.28\ ksi)(504\ in.^2)} \quad \text{(Eq. iii)}$$

$= -3.89$ in.

Again, no reinforcement is required. Check the minimum spacing required by Equation ii:

$$s > \frac{12A_{vf}f_y}{0.05A_{cv}} \qquad \text{(Eq. ii)}$$

$$> \frac{12(0.4\text{ in.}^2)(60\text{ ksi})}{0.05(504\text{ in.}^2)}$$

$$> 11.4\text{ in.}$$

Since the minimum amount of reinforcement required by Equations ii and iii is zero, no reinforcement is required for interface shear transfer.

Determine Controlling Transverse Reinforcement Spacing

The required reinforcement spacing for strength is 29.0 in.
The maximum permitted reinforcement spacing is 24 in.
The minimum reinforcement area is based on a spacing of 47.8 in.
No reinforcement is required for interface shear transfer.

The controlling spacing at the critical section for shear is 24 in.

Longitudinal Reinforcement *(5.8.3.5)*

$$\frac{d_v(1+\cot\theta)+x_{face}}{L} < 0.2$$

Where:

d_v = 73.86 in.
θ = 29.60°
L = 1680 in.
x_{face} = 15 in.

$$\frac{d_v(1+\cot\theta)+x_{face}}{L} = \frac{73.86\text{ in.}(1+\cot 29.6^\circ)+15\text{ in.}}{1680\text{ in.}} = 0.13$$

Therefore, the longitudinal reinforcement requirement needs not be checked. However, for completeness, calculations are shown below.

Critical Section:

Assuming $N_u = 0$, Eq. 5.8.3.5-1 becomes:

$$A_s f_y + A_{ps} f_{ps} \geq \frac{|M_u|}{d_v \phi_f} + \left(\left| \frac{V_u}{\phi_v} - V_p \right| - 0.5 V_s \right) \cot\theta$$

Where:

A_s = 10.48 in.^{2}

f_y = 60 ksi

A_{ps} = 2.448 in.^{2}

f_{ps} = 253.66 ksi

$|M_u|$ = 21428.4 k-in.

d_v = 73.86 in.

ϕ_f = 0.9 (see moment calculations)

$|V_u|$ = 276 k

ϕ_v = 0.9

V_s = $\frac{A_v f_y d_v \cot\theta}{s}$, not to be taken as larger than $\frac{|V_u|}{\phi_v}$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

d_v = 73.86 in.

θ = 29.6°

s = 11.4 in.

$$\frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.40 \text{ in.}^2)(60 \text{ ksi})(73.86 \text{ in.})(\cot 29.6^\circ)}{11.4 \text{ in.}} = 273.72 \text{ k}$$

$$\frac{|V_u|}{\phi_v} = \frac{|276\text{ k}|}{0.9} = 306.67\text{ k}$$

V_s = 273.72 k

V_p = 41.06 k

θ = 29.6°

$$A_s f_y + A_{ps} f_{ps} = (10.48\text{ in.}^2)(60\text{ ksi}) + (2.448\text{ in.}^2)(253.66\text{ ksi}) = 1275.6\text{ k}$$

$$\frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta =$$

$$= \frac{21428.4\text{ k}-\text{in.}}{(73.86\text{ in.})(0.9)} + (|306.67\text{ k} - 41.06\text{ k}| - 0.5(273.72\text{ k}))\cot(29.6^\circ)$$

$$= 548.6\text{ k}$$

1275.6 k > 548.6 k OK

Face of Pier:

$|M_u|$ has been calculated to be 36770.4 k-in. at the pier face.

$|V_u|$ has been calculated to be 290.8 k at the pier face.

$$A_s f_y + A_{ps} f_{ps} \geq \frac{|M_u|}{d_v \phi_f} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

Where:

A_s = 10.48 in.^{2}

f_y = 60 ksi

A_{ps} = 2.448 in.^{2}

f_{ps} = 253.66 ksi

$|M_u|$ = 36770.4 k-in.

d_v = 73.86 in.

ϕ_f = 0.9 (see moment calculations)

$|V_u|$ = 290.8 k

ϕ_v = 0.9

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}\text{, not to be taken as larger than } \frac{|V_u|}{\phi_v}$$

Where:

A_v = 0.40 in.^{2}

f_y = 60 ksi

d_v = 73.86 in.

θ = 29.6°

s = 11.4 in.

$$\frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.40\text{ in.}^2)(60\text{ ksi})(73.86\text{ in.})(\cot 29.6^\circ)}{11.43\text{ in.}} = 286.69\text{ k}$$

$$\frac{|V_u|}{\phi_v} = \frac{|290.8\text{ k}|}{0.9} = 323.11\text{ k}$$

V_s = 273.72 k

V_p = $A_{ps}^{harped} f_{px} \sin\Psi$

$$f_{px} = \frac{f_{pe} l_{px}}{60 d_b} \qquad \text{(Eq. 5.11.4.2-2)}$$

$$= \frac{(167.63\text{ ksi})(15\text{ in.})}{60(0.5\text{ in.})}$$

= 83.82 ksi

V_p = $(2.448\text{ in.}^2)(83.82\text{ ksi})\sin 4.47^\circ$

= 15.99 k

θ = 29.6°

$$A_s f_y + A_{ps} f_{ps} = (10.48\text{ in.}^2)(60\text{ ksi}) + (2.448\text{ in.}^2)(253.66\text{ ksi}) = 1249.8\text{ k}$$

$$\frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left| \frac{V_u}{\phi_v} - V_p \right| - 0.5V_s \right) \cot\theta =$$

$$= \quad \frac{36770.4 \text{ k} - \text{in.}}{(73.86 \text{ in.})(0.9)} + \left(\left| \frac{290.8 \text{ k}}{0.9} - 15.99 \text{ k} \right| - 0.5(273.72 \text{ k}) \right) \cot(29.6^\circ)$$

$$= \quad 852.9 \text{ k}$$

1249.8 k > 852.9 k OK

Check Calculated Capacity at the Critical Section Against Eq. 5.8.3.3-2

V_n shall be the lesser of $V_n = V_c + V_s + V_p$ and

$$V_n = 0.25f'_c b_v d_v + V_p \qquad \text{(Eq. 5.8.3.3-2)}$$

Where:

f'_c = 7 ksi

b_v = 6 in.

d_v = 73.86 in.

V_p = 41.06 k

V_n = 0.25(7 ksi)(6 in.)(73.86 in.) + 41.06 k

= 816.5 k

$V_c + V_s + V_p$ = 157.83 k + 286.69 k + 41.06 k = 485.6 k

816.5 k > 485.6 k

Equation 5.8.3.3-2 does not control. The controlling spacing for strength at the critical section for shear is based on $V_c + V_s + V_p$.

Tabulated Required Maximum Spacings

Pt. along span 1	x (ft.)	s (in.)	s (in.)	s (in.)	s (in.)	s (in.)
		5.8.3.3-4	5.8.2.5-1	5.8.2.7	Eq.'s i-iii	Governing
Critical Pt.	6.05	28.4	47.8	24.0	N/A	24.0
0.1	14.00	17.0	47.8	24.0	N/A	17.0
0.2	28.00	16.3	47.8	24.0	N/A	16.3
0.3	42.00	5367.0	47.8	24.0	N/A	24.0
0.4	56.00	24925.2	47.8	24.0	N/A	24.0
0.5	70.00	24829.2	47.8	24.0	N/A	24.0
0.6	84.00	73.6	47.8	24.0	N/A	24.0
0.7	98.00	27116.6	47.8	24.0	N/A	24.0
0.8	112.00	20.8	47.8	24.0	N/A	20.8
0.9	126.00	27.3	47.8	24.0	N/A	24.0
Critical Pt.	133.95	29.3	47.8	24.0	N/A	24.0

Pt. along span 2	x (ft.)	s (in.)	s (in.)	s (in.)	s (in.)	s (in.)
		5.8.3.3-4	5.8.2.5-1	5.8.2.7	Eq.'s i-iii	Governing
Critical Pt.	6.05	13.8	47.8	24.0	N/A	13.8
0.1	10.00	16.3	47.8	24.0	N/A	16.3
0.2	20.00	124.4	47.8	24.0	N/A	24.0
0.3	30.00	4443.7	47.8	24.0	N/A	24.0
0.4	40.00	138.4	47.8	24.0	N/A	24.0
0.5	50.00	25805.3	47.8	24.0	N/A	24.0
0.6	60.00	26086.6	47.8	24.0	N/A	24.0
0.7	70.00	4674.4	47.8	24.0	N/A	24.0
0.8	80.00	30687.8	47.8	24.0	N/A	24.0
0.9	90.00	189.1	47.8	24.0	N/A	24.0
Critical Pt.	93.95	93.9	47.8	24.0	N/A	24.0