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

This design guide focuses on the Load and Resistance Factor Design (LRFD) of Precast Prestressed Concrete (PPC) I-beams, Bulb T-beams, and IL-beams. The design procedure is presented first followed by a worked example. All article and equation references are to the AASHTO LRFD Bridge Design Specifications, 8th Edition, unless noted otherwise.

The Illinois Department of Transportation (IDOT) utilizes standard beam shapes ranging in depth from 27 inches to 90 inches. These shapes are in two main categories. The terms “I-beam” and “Bulb T-beam” refer to shapes based on the AASHTO standard shapes. These shapes range from 36 in. minimum depth to 72 in. maximum depth. The term “IL-beam” refers to newer shapes that were introduced via All Bridge Designers (ABD) Memoranda 15.2 and 21.2. These shapes range in depth from 27 in. to 90 in., and have been engineered to allow for longer span lengths than the previous sections without an increase in beam depth.

IL-beams with 81 and 90 inch depths also have been developed. These shapes are able to accommodate simple spans up to 189 ft. in length. For longer beams, site location and delivery route are a concern and shall be considered in both planning and design.

Beam selection charts and standard strand patterns for LRFD designs have been developed and are provided in Bridge Manual (BM) Sections 2.3 and 3.4 and All Bridge Designers Memoranda 15.2 and 21.2. 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 for I-Beams and Bulb-T Beams, and in ABD 15.2 and 21.2 for IL-Beams.

The following calculations are required, and will be outlined in this design guide: section properties, loading, strand pattern selection, prestress losses, strand development lengths, strand group eccentricities, temporary beam stresses, final beam stresses and resisting moment capacities, and negative moment reinforcement design (multi-span bridges only). A numeric example follows. For ease of navigation, bookmarks have been created for each design check.

The sign convention used for stresses in the example labels negative results with “tension” and positive results with “compression” unless otherwise noted.

The AASHTO LRFD Bridge Design Specification formulas typically contain a lightweight concrete factor, λ. This factor is 1.0 for all PPC beams, and it is not repeated in this design guide.

LRFD Design Procedure and Equations

Section Properties

Beam Selection

The Type, Size, and Location (TSL) Report will provide a required beam depth. This beam depth correlates with minimum clearances and hydraulic openings and is required to be used. For many of the beam depths, there are different options for the shapes to be used. The shapes fall into two categories:

- I-Beams and Bulb T-Beams, which are shapes based on standard AASHTO sections. These sections use 0.5 inch diameter strands and have concrete compressive strengths of 6 ksi or 7 ksi, and have been utilized in Illinois for many years.

- IL-Beams were designed to span longer distances and have larger allowable beam spacings for the same span lengths as I- and Bulb-T beams. They also have additional improvements such as curved forms to improve concrete consolidation and wider flanges for greater stability. These shapes were introduced with ABD 15.2.

In general, IL-sections should be used on new structures, with the lightest IL-section being the preferred section to be used. For beam replacements and structure widenings, I-Beams and Bulb-T beams should be used, as they have the similar stiffnesses to those beams already present on the structure.

Non-Composite Beam Section Properties (BM 3.4.4, ABD 15.2)

Precast prestressed I-, Bulb-T, and IL-beams are noncomposite and simply supported for all load conditions prior to hardening the slab. Non-composite beam section properties are found in Bridge Manual Section 3.4 for I- and Bulb-T beams, and ABD 15.2 for IL-beams.

Moduli of Elasticity (5.4.2, 5.4.3, 5.4.4)

The modulus of elasticity for prestressing steel (E_p) and the modulus of elasticity for mild steel (E_s) are specified by AASHTO as follows:

E_p = 28500 ksi (5.4.4.2)

E_s = 29000 ksi (5.4.3.2)

The modulus of elasticity for the concrete in the composite section is calculated three times: for the initial (release) strength of the beam concrete (E_{ci}), the 28-day strength of the beam concrete ($E_{c,beam}$), and the 28-day strength of the concrete slab ($E_{c,slab}$). E_{ci} is not used in section property calculations, but is used later in the design when calculating lifting stresses and beam camber.

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

$E_{ci} = 120{,}000 K_1 w_{ci}^2 f'^{\,0.33}_{ci}$ (Eq. 5.4.2.4-1)

$E_c = 120{,}000 K_1 w_c^2 f'^{\,0.33}_c$ (Eq. 5.4.2.4-1)

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 (Table 3.5.1-1)

= 0.145 for normal weight concrete with $f'_c \le 5.0$ ksi

= $0.140 + 0.001f'_c$ for normal weight concrete with 5.0 ksi < $f'_c \le 15.0$ ksi

f'_c = specified 28-day compressive strength of concrete for use in design (ksi)

= 4 ksi for slab concrete

= 6 ksi or 7 ksi for I- and Bulb-T beams, depending upon strand pattern. See Fig. 3.4.4.1-1 and Tables 3.4.4.1-12 of the Bridge Manual. Most strand patterns require 6 ksi concrete. Those marked “H” require 7 ksi concrete.

= 8.5 ksi for IL-beams

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

= 5 ksi for concrete with f'_c equal to 6 ksi

= 6 ksi for concrete with f'_c equal to 7 ksi

= 6.5 ksi for concrete with f'_c equal to 8.5 ksi

Modular Ratio

The slab and beam have different moduli of elasticity, therefore, a section transformation of either the beam or the slab is required in order to correctly analyze a composite section. Whether the slab properties are transformed to be consistent with the beam properties, or vice versa, is dependent on what stresses are being calculated.

When calculating beam stresses, the beam properties are used. The slab properties must be transformed by a modular ratio to be consistent with the beam properties. The modular ratio used is:

$$n = \frac{E_{c,slab}}{E_{c,beam}}$$

When calculating slab stresses in an uncracked section, the slab properties are used. The beam properties must be transformed by a modular ratio equal to the reciporocal of the equation above.

Composite Section Properties

Effective Flange Width **(4.6.2.6)**

If the beams and overhangs are sized appropriately, the interior girder is expected to control the design of the beam over the exterior girder. If this is the case, the effective flange width is taken as the beam spacing. In the event that the exterior girder controls the design, the effective flange width is taken as one-half the beam spacing plus the width of the overhang.

Fillets

Concrete fillet heights are dependent on beam camber and therefore are not known until the beam is designed. For load calculations, a fillet of 2 inches was used to determine the standard strand patterns. Whether this fillet is included in the composite section properties is left to the discretion of the designer. A common conservative assumption is to assume the fillet has zero height, but still contributes 2 inches of concrete dead load weight.

Positive Moment Composite Design Section

When the deck hardens, a composite section is used to calculate stresses and capacities for loads superimposed on the slab, including parapet loads, future wearing surface loads, and live loads. The composite positive moment design section consists of a concrete slab and a concrete beam element, with the slab transformed to the beam properties. Compression reinforcement in the deck is not included in section properties.

Negative Moment Composite Design Section

For multi-span girders, continuity diaphragms allow for continuous, composite behavior and negative moment regions over/near supports. The slab concrete in the negative moment region is assumed to be in tension and is therefore not effective. For the purpose of calculating section properties, the beam itself is assumed to be uncracked. The composite negative moment design section consists of the beam concrete and the slab reinforcement.

Loading

Span Lengths of Noncomposite Beams and Composite Sections

For non-composite loading (beam and slab), the beams are simply supported, with the span lengths equal the the centerline-to-centerline bearing distance.

For composite loading (parapet, wearing surface, live loads, plus other appurtenances if present), the span length is measured using the centerlines of bearings at abutments and the centerlines of piers. For multispan structures, this results in the composite span length being different from the non-composite span length because the centerlines of the bearings at the piers are offset from the centerline of the pier.

For integral abutments, the centerlines of bearings are colinear with the centerline of abutment (see Figures 36 and 37 in ABD 15.2). At piers, the distance from the centerline of pier to centerline of bearing is given in Figure 34 in ABD 15.2.

Non-Composite (DC1) Dead Loads and Composite (DC2, DW) Dead Loads

As per C3.5.1 of the AASHTO code, the unit weight of reinforced concrete is generally taken as 0.005 kcf greater than the unit weight of plain concrete. Typically, 0.150 kcf is assumed as the unit weight of all concrete, despite the fact that, according to AASHTO, concrete weight will vary from 0.145 kcf to 0.1485 kcf for the various strengths of concrete in a section.

Beam weights per foot (DC1) are given in Bridge Manual Table 3.4.4.2-1 and in Figure 1 of ABD 15.2.

Fillet thicknesses (DC1) are typically assumed to be 2 in. for the purpose of load calculations.

The tributary width for the slab weight (DC1) is taken as the effective flange width.

Constant-slope parapets have areas of 3.5 ft.^{2} / ft. for 39 in. tall parapets and 3.8 ft.^{2} / ft. for 44 in. tall parapets. This results in weights per foot (DC2) of 0.525 k/ft. for 39 in. tall parapets and 0.570 k/ft. for 44 in. tall parapets.

The design allowance for future wearing surface weight per square foot (DW) is given on the TSL plan. It is typically applied to the effective flange width for a beam, unless a sidewalk or median is present.

<u>Live Load Distribution Factors</u> <u>(4.6.2.2)</u>

Live load distribution factors are used to determine what proportion of a loaded lane is resisted by a typical beam. There are three different live load distribution factors used in design: moment, shear and reaction, and deflection.

There are separate live load distribution factors for each positive and negative moment region in the structure. The live load distribution factors are dependent upon the length of span for which they are being calculated. For negative moment regions, live load distribution factors are based off the average length of the two spans contributing to the negative moment region.

When calculating live load distribution factors, both the single-lane and multi-lane distribution factors are calculated, with the controlling factor used in the design. Depending upon the layout of the beams and overhangs, there may be cases where a single truck causes greater force effects on a beam than multiple trucks at the same location, and the single-lane distribution factor would therefore control the design.

The live load distribution factors in Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1 include the possibility of multiple trucks contributing to the loading i.e. multiple presence. Fatigue loading depends upon a single-lane loaded, without multiple presence. Therefore, to calculate a fatigue live load distribution factor, the single-lane live load distribution factor is calculated, then the multiple presence factor is divided out.

Live load distribution factors will be calculated using the methods in AASHTO 4.6.2.2, without any simplifications.

Longitudinal Stiffness Parameter K_g (4.6.2.2.1)

The longitudinal stiffness parameter, K_g, is used in calculation of the moment and shear live load distribution factors.

$$K_g = n\left(I + Ae_g^{\,2}\right) \qquad \text{(Eq. 4.6.2.2.1-1)}$$

Where:

n = modular ratio of beam to deck. Note that this is the reciporocal of "n" calculated above.

$= \dfrac{E_{c,beam}}{E_{c,deck}}$

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

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

e_g = distance from centroid of deck to centroid of beam

= y'_t + fillet (if used) + $0.5t_{slab}$

Live Load Distribution Factor, Moment, Interior Beams (Table 4.6.2.2.2b-1)

g_1 = single-lane live load distribution factor

$$= 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

g_m = multiple-lane live load distribution factor

$$= 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

Where:

S = beam spacing (ft.)

L = composite span length (ft.)

t_s = slab thickness (in.)

K_g = longitudinal stiffness parameter (in.^{4})

As per Article 3.3.1 of the Bridge Manual, the skew correction factor for moment found in Table 4.6.2.2.2e-1 of the AASHTO Code shall not not applied.

$g_{fatigue}$ = live load distribution factor for fatigue limit states

= $\frac{g_1}{m}$, where m is the multiple presence factor of 1.2 for single-lane loading

(3.6.1.1.2)

Live Load Distribution Factor, Shear and Reaction, Interior Beams

(Table 4.6.2.2.3a-1)

$$g_1 = 0.36 + \frac{S}{25.0}$$

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

Skew correction = $1 + 0.2\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}(\tan\theta)$ (Table 4.6.2.2.3c-1)

S = span length (ft.)

L = composite span length (ft.)

t_s = slab thickness (in.)

K_g = longitudinal stiffness parameter (in.^{4})

θ = skew (degrees)

Live Load Distribution Factor, Deflection

$$g_{defl} = m\left(\frac{N_L}{N_b}\right)$$

Where:

m = multiple presence factor (Table 3.6.1.1.2-1)

N_L = number of lanes

= integer part of the ratio w/12.0, where w is the clear roadway width in feet between barriers (3.6.1.1.1)

N_b = number of beams

Moment and Shear Envelopes

Calculate the moment and shear envelopes using software written for this task. The short-term composite positive moment section properties are used to generate live load moments, shears, and reactions.

Strand Pattern Selection

Standard Strand Patterns

The planning selection charts located in BM Section 2.3.6.1.3 and ABD Memorandum 15.2 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 for I- and Bulb-T shapes and Figures 17 through 27 of ABD 15.2 for IL-beam shapes.

The noncomposite beam span length is used to enter the planning and design charts to determine the applicable beam sizes and strand patterns.

For IL-beams, the standard strand patterns given in ABD 15.2 were developed for simply-supported spans, but will typically also be adequate for multi-span applications, given that some additional allowances are provided:

- Multi-span bridges will have smaller positive live load moments than their simply-supported counterparts, by an amount of around 10% - 20% depending upon the span configuration. Therefore, when choosing a strand pattern for a multi-span structure, the designer should enter the charts using a reduced span length of 90%-95% ($\sqrt{80\%} - \sqrt{90\%}$) of the actual span, in order to obtain a strand pattern consistent with the anticipated lower moments.

- Use of the minimum strand pattern from the charts for a multi-span application may not be adequate due to the differences in design moments between simply-supported and continuous structures.Typically, there are several standard strand patterns to choose from for a given span length. A standard pattern in the middle of the group, with more strands than the minimum, will likely result in the most robust design. The cost of additional strands is marginal in comparison to the cost of the beam.

- For extreme span lengths and span ratios, there may be multi-span applications where none of the standard strand patterns are adequate. Designers should make every attempt to use the standard strand patterns, but, if none are found to be adequate, a user-defined pattern may be developed in accordance with the provisions below in the section “User-Defined Strand Patterns.”

The numeric example in this design guide will illustrate a procedure for choosing a standard strand pattern for multi-span applications.

User-Defined Strand Patterns

As stated above, there may be cases where the standard strand patterns given in the selection charts in BM Section 3.4 and ABD 15.2 are inadequate. In these cases, a user-defined strand pattern is required. When a user-defined strand pattern is used, the provisions of Article 5.9.4 of the AASHTO Code are required to be met. Article 5.9.4 of the AASHTO Code gives guidance on strand spacing, strands per row, debonding locations, etc. In addition to these specifications, the following is required for beams fabricated for use in Illinois:

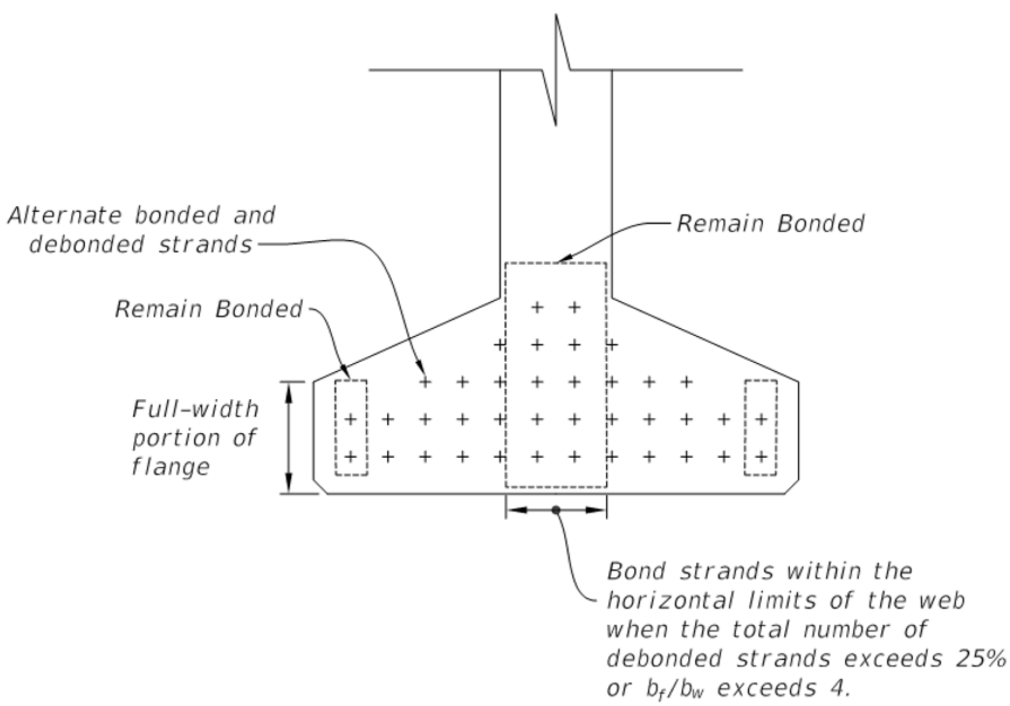
IDOT standard base sheets give permissible strand locations that are consistent with the fabrication process. These are found in View D-D of the standard base sheets for IL-beams. Movement of strands to locations that vary from those shown on the base sheets is not allowed.

For I- and Bulb-T beams, the standard strand patterns for multiple spans have been developed to account for typical span configurations. However, due to the large number of loading cases, it may be necessary to drape additional strands. Debonded strands are not allowed in these beam shapes. For I- and Bulb-T beams, the maximum number of draped strands is 16.

For IL-beams, the maximum number of draped strands is generally limited to six (6). Stress reductions should be accounted for by debonding additional strands prior to draping additional strands.

Article 5.9.4.3.3 of the AASHTO Code gives permissible strand locations for debonding. When choosing locations of debonded strands, if the rules given in the following figure are followed, then all of the provisions in Article 5.9.4.3.3 will be met.

For I- and Bulb-T beams, debonding is not allowed.



Detail of Strand Pattern on Plans

Figure 3 of ABD 15.2 gives an example of how strand patterns should be detailed on final plans. All locations of debonded strands within the member, debonding locations along the beam, and harped strands should be fully detailed such that fabricators can replicate the assumptions used to design the beams.

Prestress Losses *(5.9.3)*

Due to prestress losses, the amount of prestressing stress used to calculate service stresses in a beam is less than the ultimate strength of the strands. There are two components contributing to prestress losses that are calculated to determine these losses, elastic shortening losses (Δf_{pES}), and long-term losses (Δf_{pLT}).

Total Loss of Prestress (5.9.3.1)

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \quad \text{(Eq. 5.9.3.1-1)}$$

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)

The final stress in the strands, after losses, is required to be less that $0.8f_{py}$ (Table 5.9.2.2-1), where f_{py} is taken as $0.9f_{pu}$ (Table 5.4.4.1-1).

Instantaneous Losses (5.9.3.2)

The AASHTO Code contains provisions for losses due to anchorage set (5.9.3.2.1), friction at hold-down devices (5.9.3.2.2), and elastic shortening (5.9.3.2.3). The procedures used by prequalified prestressed concrete producers in Illinois will preclude any appreciable losses due to anchorage set and friction at hold-down devices. Therefore, only elastic shortening losses are required to be considered for prestressed beams produced in accordance with IDOT fabrication requirements.

Instantaneous elastic shortening losses (Δf_{pES}) occur when the beam changes shape due to the sudden increase in concrete stress when the strands are cut (i.e. at the time of transfer) for a prestressed concrete beam. Designers should note that, due to a strand's placement in the beam and the redistribution of stress at the time of transfer, elastic gains may be observed in those strands. It is the Department's policy to consider elastic losses only. Elastic gains are not considered.

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{(Eq. 5.9.3.2.3a-1)}$$

Where:

E_p = modulus of elasticity of prestressing steel (ksi)

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

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

$$= \frac{F_t}{A} + \frac{F_t e^2}{I} - \frac{M_b e}{I}$$

In which:

F_t = total prestressing force immediately after transfer (kips). This value is calculated by taking an initial estimate based upon a percentage of the total prestressing force prior to transfer, F_i, then that estimate is checked (C5.9.3.2.3a)

= $0.9(F_i)$ for initial iteration

F_i = total prestressing force prior to transfer (kips)

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

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

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

= 202.3 ksi for 0.6 in. diameter strands and 201.96 ksi for 0.5 ksi diameter strands. Note that these are industry-specific values, and vary slightly from the $0.75f_{pu}$ limit given in Table 5.9.2.2-1 of the AASHTO Code. For IL-beams, 0.6 in. diameter strands are used. For older PPC I- and Bulb-T shapes, 0.5 in. diameter strands are used.

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

M_b = bending moment due to beam self weight (kip-ft.). It is assumed that, once the strands are cut, the beam will camber immediately and rest on its ends in a simply-supported position. Therefore, the entire length of the beam is used to calculate the bending moment using $0.125w\ell^2$.

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

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

Using the assumption that $F_t = 0.9F_i$, Δf_{pES} is calculated, and the assumption is checked using the following formula:

$$\frac{F_t}{F_i} \approx \frac{\left(f_{pbt} - \Delta f_{pES}\right)}{f_{pbt}}$$

If the ratio of F_t / F_i is substantially different than 0.9, then reiterate using the value obtained until convergence.

Approximate Estimate of Time-Dependent Losses (5.9.3.3)

Time-dependent losses (Δf_{pLT}) are long-term losses due to shrinkage and creep of concrete and strand relaxation. The AASHTO Code provides two methods for calculating these losses. The first, given in 5.9.3.3, is an approximate method that is considered to provide reliable results by the Department. The second, given in 5.9.3.4, is a refined procedure that requires several in-depth parameters not typically available to designers, and is not used by the Department.

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(Eq. 5.9.3.3-1)}$$

In which:

γ_h = $1.7 - 0.01H$ (Eq. 5.9.3.3-2)

γ_{st} = $\dfrac{5}{(1+f'_{ci})}$ (Eq. 5.9.3.3-3)

Where:

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

= f_{pbt}, or 202.3 ksi for 0.6 in. diameter strands or 201.96 ksi for 0.5 in. diameter strands

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

A_g = 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)

Strand Development Lengths *(5.9.4.3.2)*

Because debonded strands are present, different strand groups will have strands at different stages of development within them. Calculating the strand group prestressing forces and eccentricities requires knowledge of each strand's percent development. Strand development lengths are used to calculate the percent development of strands along the length of the beam.

Strand development lengths are dependent upon parameters such as distance to neutral axis and percent development, which themselves are dependent upon strand development. Therefore, calculation of strand development is technically recursive and iteration is required to calculate an exact value. However, the two parameters above do not change greatly in the region close to strand development, and therefore development lengths do not change greatly from one iteration to the next. For that reason, one iteration is typically accurate enough to determine the strand development lengths for the strands in the beam.

Final development lengths range from around 11 feet on shallower beams to around 12 feet on deeper beams. An initial location can be estimated by comparing the depth of the beam in consideration to these values and choosing an approximate location.

$$\ell_d = K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \qquad \text{(Eq. 5.9.4.3.2-1)}$$

Where:

K = Variable to account for differences in strand fabrication

= 1.6 for members with a depth greater than 24 inches (5.9.4.3.2)

= 2 for debonded strands (5.9.4.3.2)

f_{pe} = Effective stress in prestressing steel after total losses (ksi)

= f_{pbt} * (1- % losses)

d_b = Diameter of prestressing steel strand (in.)

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

$$= f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(Eq. 5.6.3.1.1-1)}$$

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

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{(Eq. 5.6.3.1.1-2)}$$

Where f_{py} is the yield strength of the prestressing steel, taken as $0.9f_{pu}$ (Table 5.4.4.1-1).

= 0.28 for low relaxation strands.

c = Distance from extreme compression fiber to neutral axis, calculated assuming any debonded strands are not present in the strand group at the location of development length (in.)

c, neutral axis in slab

$$= \frac{A_{ps}f_{pu}}{\alpha_1 f'_{c,slab}\,\beta_1 b_{slab} + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-4)}$$

c, neutral axis in top flange of beam

$$= \frac{A_{ps}f_{pu} - \alpha_1 f'_{c,slab}\left(b_{slab} - b_{tf}\right)t_{slab}}{\alpha_1 f'_{c,slab}\,\beta_1 b_{tf} + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

c, neutral axis in web of beam

$$= \frac{A_{ps}f_{pu} - \alpha_1 f'_{c,slab}\left(b_{tf} - b_w\right)\left(t_{slab} + t_{tf}\right) - \alpha_1 f'_{c,slab}\left(b_{slab} - b_{tf}\right)t_{slab}}{\alpha_1 f'_{c,slab}\,\beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

In these equations:

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

β_1 = stress block factor

= $0.65 \le 0.85 - 0.05\left(f'_c - 4.0\right) \le 0.85$ (5.6.2.2)

A_{ps} = effective area of prestressing steel at location of development length (in.^{2})

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

α_1 = stress block factor

= 0.85 for design compressive strength not exceeding 10 ksi (5.6.2.2)

$f'_{c,slab}$ = specified compressive strength of slab concrete (ksi)

b_{slab} = untransformed effective flange width (in.)

b_{tf} = transformed beam top flange width (in.)

= $\frac{E_{c,beam}}{E_{c,slab}}$ (untransformed top flange width)

b_w = transformed beam web width (in.)

= $\frac{E_{c,beam}}{E_{c,slab}}$ (untransformed web width)

t_{slab} = slab thickness (in.)

t_{tf} = least thickness of top flange of beam (in.)

k = 0.28 for low-relaxation strands (Table C5.7.3.1.1-1)

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

Strand Group Eccentricities

Design Section Locations

Strand group eccentricities are required to be calculated in order to calculate concrete stresses and flexural capacities at each location where these checks are required. These checks are required at locations of maximum moment, each debond location, as well as one transfer length beyond each debond location (for standard strand patterns, debond locations are all one transfer length apart, making the "debond location 1 + transfer" location equal to debond location 2, etc. Therefore, strand eccentricities will be required at each of the following locations, when applicable:

- One transfer length from end of beam (for negative moment region checks)
- Critical Section for Shear
- Centroid of lifting loops
- Debond location 1
- Debond location 1 + transfer / Debond location 2
- Debond location 2 + transfer
- 0.4L, 0.5L, Point of maximum positive moment, if different from 0.4L or 0.5L (all strands should be fully developed by the harping point, so these points will all have the same strand group eccentricity)

Percent Development of Strands (5.11.4)

Calculating the eccentricity of a strand group is essentially a centroid problem, with the additional complication that not all of the strands are fully developed at each location. The percent development of each strand is calculated, and that percent development is used to calculate an effective area of steel for each strand. These effective areas are then used to calculate the centroid of the strands group, and, in turn, the eccentricity of the strand group.

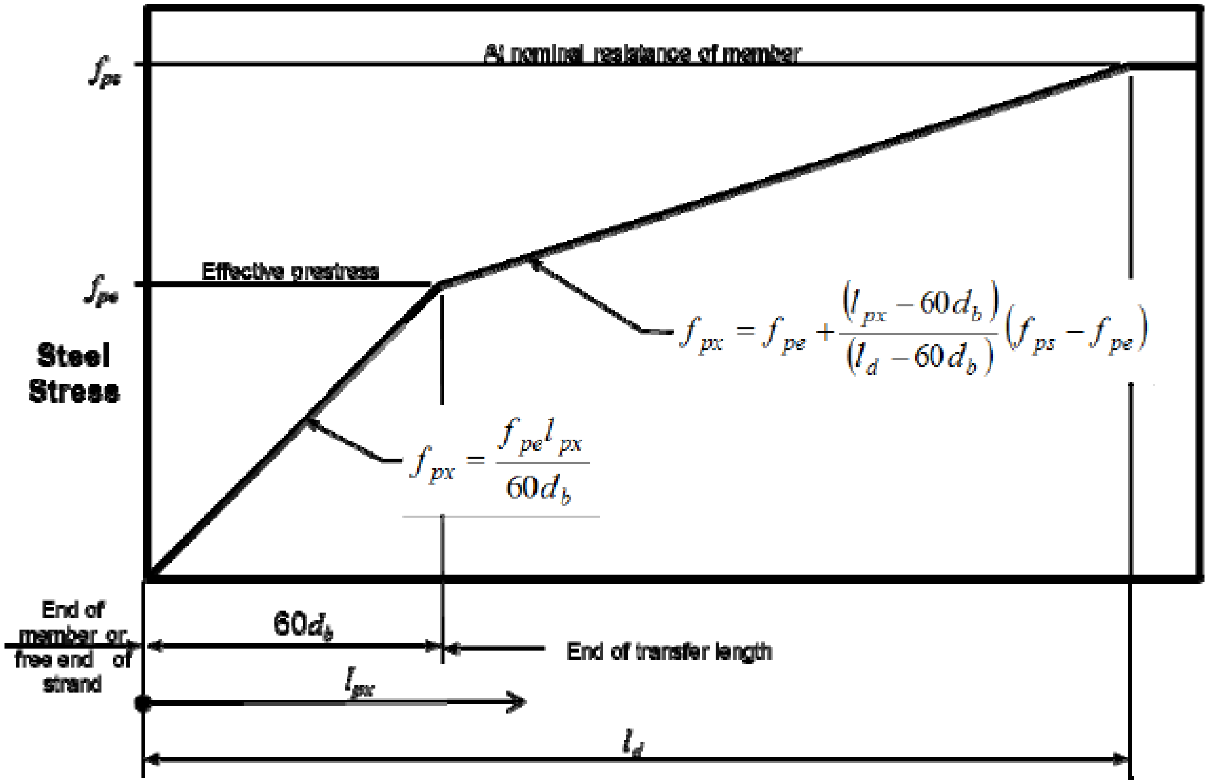


Figure C5.9.4.3.2-1—Idealized Relationship between Steel Stress and Distance from the Free End of Strand

The transfer length is taken as 60 strand diameters from the point of bonding of the strand (5.9.4.3).

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

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

$$f_{px} = \frac{f_{pe}\ell_{px}}{60d_b} \qquad \text{(Eq. 5.9.4.3.2-2)}$$

For sections between the end of the transfer length and the end of the development length, f_{px} is linearly interpolated using the following equation.

$$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{\ell_d - 60d_b}\left(f_{ps} - f_{pe}\right) \quad \text{(Eq. 5.9.4.3.2-3)}$$

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.)

Temporary Stresses *(5.9.2.3.1)*

Temporary stresses are checked 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.

<u>Lifting Loop Locations</u>

Lifting loop locations are dependent upon the length of the beam and the weight of the beam. The required number and location of required lifting loops is given in Figure 3.4.7-1 in the Bridge Manual and Figure 28 in ABD 15.2.

<u>Temporary Stress Limits</u> <u>(5.9.2.3.1)</u>

Compressive stresses shall be limited to $0.65f'_{ci}$. (5.9.2.3.1a)

Tensile stresses shall be limited to those specified in Table 5.9.2.3.1b-1. For prestressed beams in Illinois, beams have bonded reinforcement in all areas (i.e. there are no areas where all strands are debonded), and will be designed to resist a tensile force equal to $0.24\sqrt{f'_{ci}}$. (5.9.2.3.1b)

Load Conditions for Temporary Stresses

There are three support conditions to consider prior to utilization of 28-day concrete strength in calculations.

- The first temporary condition occurs when the strands are released and the beam is still setting on the prestressing bed. During this time, the beam will camber up and bear on its extreme ends.
- The second temporary condition occurs when lifting the beam out of the prestressing bed. During this time, the beam is supported from above by its lifting loops.
- The third temporary condition occurs when placing the beam in temporary storage at the fabrication plant. During this time, the beam is set on donnages, which are temporary supports placed by the fabricator.

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 fabrication policies are followed. Therefore, the condition of lifting the concrete beams from the prestressing bed is the only temporary condition checked.

For the load condition where the beam is being lifted out of the bed, stresses need to be checked in the locations of the maximum positive moment (mid-span), maximum negative moment (centroid of lifting loops), locations of debonding, transfer locations, and harping locations.

See Figure 1 for the support and loading diagram used to calculate the dead load moments for checking temporary stresses, M_{bts}:

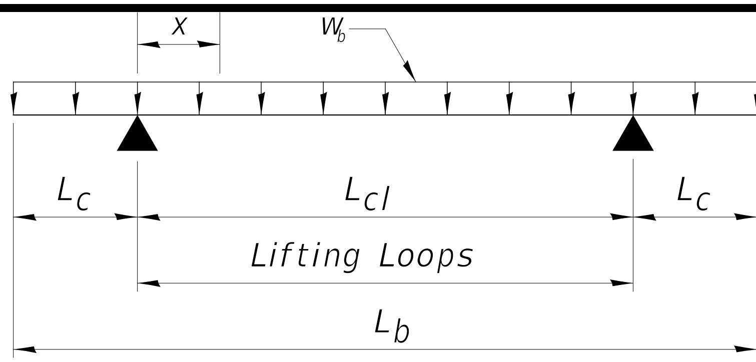


Figure 1

@ Centroid of Lifting Loops:

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

@ Any point x between lifting loops, where x is the distance from the loop:

$$M_{bts} = -\frac{w_b L_c^2}{2} + \frac{w_b x}{2}\left(L_b - 2L_c - x\right)$$

@ Center:

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

@ Harping Point (0.4 L_b):

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

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 centroid of lifting loops on one side of the beam (ft.)

L_c = length of cantilever (ft.)

L_b = total length of beam (ft.)

Calculate Temporary Stresses

@ Centroid of Lifting Loops (or any regions with negative lifting moments):

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

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

@ Center of beam ($0.5L_b$) or Harping Point ($0.4L_b$) (or any regions with positive lifting moments):

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

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

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-in.). Note that this value is typically output by software in k-ft., not k-in. It is a common error not to convert this value in calculations.

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)

Stability During Shipping and Construction

Article 5.5.4.3 of the AASHTO LRFD Bridge Design Specifications states that buckling of precast members shall be investigated for all stages of construction. All standard IDOT strand patterns have been investigated using the allowable tolerances in the IDOT Manual for Fabrication of Precast Prestressed Concrete. All beams utilizing standard strand patterns are stable for lifting, given that the standard lifting loop locations provided by IDOT are used for a given beam size and length. If a non-standard strand pattern is used, the designer is required to check for roll stability. Examples of these calculations are found in Section 8.10 of the PCI Bridge Design Manual.

As per the IDOT Manual for Fabrication Precast Prestressed Concrete, Article 3.6.4, the beam fabricator is required to evaluate the transportation loading. Shipping stress calculations are dependent upon truck axle stiffness, roadway grade, etc. Designers do not know this information when they are desigining the beams.

The erection engineer is required to check the adequacy of the beam during all phases of erection.

Positive Moment Region Design

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.

Required design checks in positive moment regions consist of checking concrete stresses for factored Service I and Service III loading (5.9.2.3.2), and verifying the flexural capacity of the section for factored Strength I loading (5.6.3.2).

Service Limit State Stresses (5.9.2.3.2)

Service limit state stresses are checked for the beam in its final placement in the structure. The concrete strength is equal to the 28-day 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 5.9.2.3.2a-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 5.9.2.3.2b-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 (5.9.2.3.2)

Compression (For Service I load combination):

$0.60\phi_w f'_c$ (a) (Table 5.9.2.3.2a-1)

$0.45f'_c$ (b) (Table 5.9.2.3.2a-1)

Tension (For Service III load combination):

$0.19\sqrt{f'_c}$ (Table 5.9.2.3.2b-1)

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:

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

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

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{(M_{DC1} + M_{DW1})}{S_b} - \frac{(M_{DC2} + M_{DW2})}{S'_b} - 0.8\frac{M_{LL+IM}}{S'_b}$$

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-in.)

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

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

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

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

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)

Fatigue Limit State Stresses (5.5.3.1)

In positive moment regions, the concrete compressive stresses due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed the limits shown below.

The section properties used for calculating the compressive stress are determined based on whether or not the concrete section is considered to be cracked.

The concrete tensile stress at the bottom of the beam due to the factored Fatigue I load combination in addition to the unfactored permanent loads and prestress is calculated and checked as follows:

$$f_b < 0.095\sqrt{f'_c} \qquad (5.5.3.1)$$

Where:

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{(M_{DC1} + M_{DW1})}{S_b} - \frac{(M_{DC2} + M_{DW2})}{S'_b} - 1.75\frac{M_{FL+IM}}{S'_b}$$

If the tensile stress exceeds $0.095\sqrt{f'_c}$, that does not mean the beam has failed. It only means that cracked section properties are required to be used in the compressive stress check.

The concrete compressive stress at the top of the beam due to the factored Fatigue I load combination in addition to one-half of the unfactored permanent loads and prestress is calculated and checked as follows:

$$f_t < 0.4f'_c \qquad (5.5.3.1)$$

Where:

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{(M_{DC1} + M_{DW1})}{S_t} + \frac{(M_{DC2} + M_{DW2})}{S'_t}\right] + 1.75\frac{M_{FL+IM}}{S'_t}$$

In which:

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-in.)

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

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

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

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

S_t = non-composite section modulus for the top fiber of the beam (in.^{3}). If the section is found to be cracked, the cracked section modulus is used.

S_b = non-composite section modulus for the bottom fiber of the beam (in.^{3}). If the section is found to be cracked, the cracked section modulus is used.

S'_t = composite section modulus for the top fiber of the beam (in.^{3}). If the section is found to be cracked, the cracked section modulus is used.

S'_b = composite section modulus for the bottom fiber of the beam (in.^{3}) If the section is found to be cracked, the cracked section modulus is used.

<u>Strength I Moment</u>

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

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> <u>(5.6.3.2.1)</u>

The flexural resistance of a concrete section is calculated using the procedure found in Article 5.6.3.2 of the AASHTO LRFD Bridge Design Specifications. Because all PPC I-, Bulb-T, and IL-beams are flanged sections, the equations used in calculating the section properties is dependent upon the depth of the neutral axis of the section. The neutral axis may be in the slab, the top flange, or the web of the section. Typically, the neutral axis is initially assumed to be in the slab, then checked. If the neutral axis is found to be at a depth greater than the slab thickness, then one of the other two assumptions will be checked depending upon its location.

The location of the neutral axis may be assumed to occur at a depth equal to the height of the equivalent rectangular stress block in the section, or "a." This depth is calculated using the following equations:

$a = \beta_1 c$

Where:

c, neutral axis in slab

$$= \frac{A_{ps} f_{pu}}{\alpha_1 f'_{c,slab} \beta_1 b_{slab} + kA_{ps} \frac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-4)}$$

c, neutral axis in top flange of beam

$$= \frac{A_{ps} f_{pu} - \alpha_1 f'_{c,slab} \left(b_{slab} - b_{tf}\right) t_{slab}}{\alpha_1 f'_{c,slab} \beta_1 b_{tf} + kA_{ps} \frac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

c, neutral axis in web of beam

$$= \frac{A_{ps} f_{pu} - \alpha_1 f'_{c,slab} \left(b_{tf} - b_w\right)\left(t_{slab} + t_{tf}\right) - \alpha_1 f'_{c,slab} \left(b_{slab} - b_{tf}\right) t_{slab}}{\alpha_1 f'_{c,slab} \beta_1 b_w + kA_{ps} \frac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

The above equations have been modified from those found in AASHTO to be more specific with respect to slab, top flange, and web dimensions.

The above equations are for a section wherein the beam dimensions have been transformed to the slab materials, hence the use of $f'_{c,slab}$ in the equations.

In these equations:

a	=	depth of equivalent rectangular stress block (in.)
c	=	distance from the extreme compression fiber to the neutral axis (in.)
β_1	=	stress block factor
	=	$0.65 \le 0.85 - 0.05\left(f'_c - 4.0\right) \le 0.85$ (5.6.2.2)

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

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

α_1 = stress block factor

= 0.85 for design compressive strength not exceeding 10 ksi (5.6.2.2)

$f'_{c,slab}$ = specified compressive strength of slab concrete (ksi)

b_{slab} = untransformed effective flange width (in.)

b_{tf} = transformed beam top flange width (in.)

= $\frac{E_{c,beam}}{E_{c,slab}}$ (untransformed top flange width)

b_w = transformed beam web width (in.)

= $\frac{E_{c,beam}}{E_{c,slab}}$ (untransformed web width)

t_{slab} = slab thickness (in.)

t_{tf} = least thickness of top flange of beam (in.)

k = 0.28 for low-relaxation strands (Table C5.7.3.1.1-1)

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

Once the neutral axis depth has been calculated, the factored flexural resistance M_r is calculated in a similar manner. There are three formulas for nominal resistance: one if the neutral axis is in the slab, one if the neutral axis is in the top flange of the beam, and one if the neutral axis is in the web.

$$M_r = \phi M_n \qquad \text{(Eq. 5.6.3.2.1-1)}$$

Where:

M_n, neutral axis in slab

$$= A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) \qquad \text{(Eq. 5.6.3.2.2-1)}$$

M_n, neutral axis in top flange of beam

$$= A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 f'_c\left(b_{slab} - b_{tf}\right) t_{slab}\left(\frac{a}{2} - \frac{t_{slab}}{2}\right) \qquad \text{(Eq. 5.6.3.2.2-1)}$$

M_n, neutral axis in web of beam

$$= A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 f'_c\left(b_{slab} - b_{tf}\right)t_{slab}\left(\frac{a}{2} - \frac{t_{slab}}{2}\right) + \alpha_1 f'_c\left(b_{tf} - b_w\right)\left(t_{slab} + t_{tf}\right)\left(\frac{a}{2} - \frac{t_{slab} + t_{tf}}{2}\right)$$

(Eq. 5.6.3.2.2-1)

In which:

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

M_n = nominal flexural resistance (kip-ft.)

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

= $f_{pu}\left(1 - k\frac{c}{d_p}\right)$ If $f_{pe} \geq 0.5\ f_{pu}$ (Eq. 5.6.3.1.1-1)

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

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

Δf_{pT}= total loss (ksi)

ϕ = resistance factor, as calculated using Eq. 5.5.4.2.1-1. The resistance factor ϕ is dependent upon the amount of strain in the reinforcement. For tension-controlled sections, which is typical for a prestressed beam, the resistance factor ϕ is 1.0.

ϕ = $0.75 \leq \phi = 0.75 + \frac{0.25\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \leq 1.0$ (Eq. 5.5.4.2-1)

ε_t = net tensile strain in the extreme tension steel at nominal resistance, calculated using the procedure found in Article C5.6.2.1

= $\frac{0.003\left(h_{beam} + t_{fillet} + t_{slab} - 2\text{ in.} - c\right)}{c}$, in which the 2 in. is the distance from the bottom of the beam to the centerline of the bottom row of tendons.

d_p = distance from the extreme compression fiber to the centroid of the tendon group (in.)

ε_{cl} = compression-controlled strain limit, taken as 0.002 for prestressed concrete sections (5.6.2.1)

ε_{tl} = tension-controlled strain limit, taken as 0.005 for prestressed concrete sections (5.6.2.1)

The equations shown above have been simplified to include only the prestressing steel. Mild steel is not present on the bottom portion of prestressed beams in Illinois, and compression steel in the top flanges is ignored.

Minimum Reinforcement (5.6.3.3)

AASHTO 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 factored Strength I moment.

$M_r \geq M_{cr}$ (5.6.3.3)

In which:

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{DC1}\left(\frac{S'_b}{S_b} - 1\right)\right] \quad \text{(Eq. 5.6.3.3-1)}$$

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

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

Where:

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 Region Design *(5.7.3)*

Precast prestressed concrete members are made continuous at interior supports using the continuity diaphragm provisions found in Article 5.12.3.3 of the AASHTO Code. Therefore, all loads applied after the deck has been hardened, including parapet, future wearing surface, and live loads, are applied to a composite beam/slab section, with negative moment regions occurring at interior supports. The continuity diaphragm calculations required in Article 5.12.3.3 are satisfied by the standard details given on base sheets, and no additional calculations are required by designers.

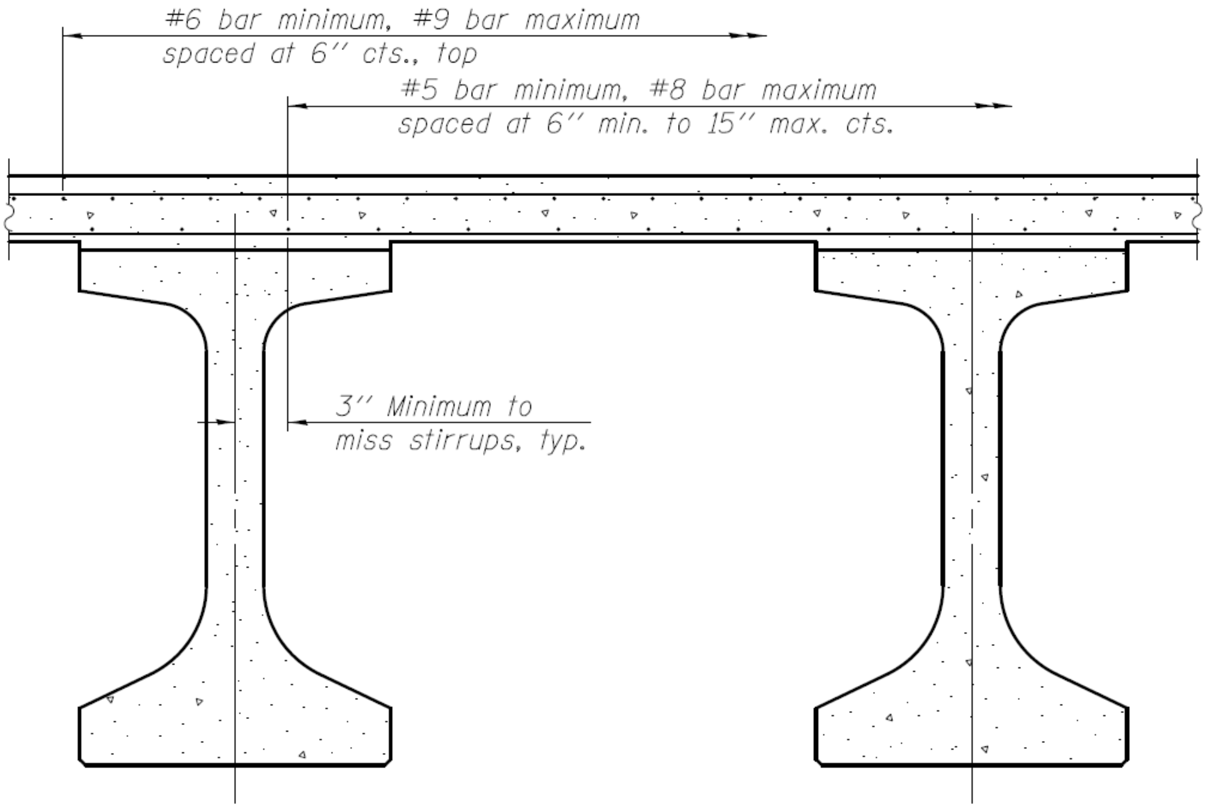
Similar to precast prestressed concrete girders in positive moment regions, girders in negative moment regions are designed to resist service stresses, fatigue stresses, and applied flexural moments. Stress checks are performed in a manner similar to those in positive moment regions. For flexural resistance, the moment-resisting section is assumed to be a non-prestressed concrete beam, wherein the longitudinal slab reinforcement and the concrete at the bottom of the bottom flange of the beam generate a resisting couple. The compressive strength is the 28 day compressive strength of the beam concrete. The effects of the non-prestressed diaphragm concrete are ignored. The top and bottom layers of longitudinal slab reinforcement are used as the main tension resisting element. Typical longitudinal reinforcement configuration including size and spacing limitations is shown in Figure 2.

The flexural capacity of the negative moment region is typically calculated first. Then, once the reinforcement in the slab is finalized, the stress checks in the beam are performed. The design procedure for the composite negative moment region consists of the following steps:

1. Calculate the maximum moments at the pier.
2. Estimate the total area of longitudinal reinforcement required in the slab at negative moment design section.
3. Determine trial reinforcement size and spacing that satisfies the total area estimate and maintains the reinforcement proportioning rules shown in Figure 2 below.
4. Check the adequacy of the reinforcement for the strength limit state, the service limit state, and the fatigue limit state.
5. Using the reinforcement calculated above, determine the composite negative moment region section modulus.
6. Check the concrete in the prestressed beam for Service I and Fatigue I checks.
7. Using the proportioning rules in Article 5.10.8.1.2, estimate an area of reinforcement required to continue past the point where a portion of the main reinforcement will be cut off.

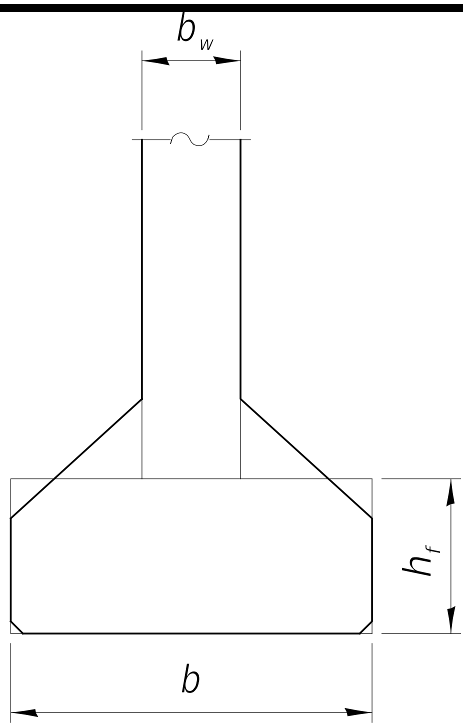
The compressive stress in the bottom fiber of the bottom flange shall be checked near the pier. The compression block area at the bottom flange of the beam has been modified as shown in Figure 3 in order to simplify the calculations. The tapered portion of the bottom flange has been converted into a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

The provisions of Article 5.12.3.3 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 have been successful in minimizing distress from these forces. The Department believes that the intent of Article 5.12.3.3 has been sufficiently addressed with the standard continuity diaphragm details 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 5.12.3.3.



CROSS SECTION
AT PIER

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"
27" IL-Beam	30"	7"	7.8"
All other IL-Beams	38"	7"	9.5"

SIMPLIFIED COMPRESSION BLOCK

Compression block assumed to act in lower 1 / 3 of tapered portion of beam. Concrete in rounded spandrels of IL-beams is conservatively ignored.

Figure 3

Calculate Strength I Moment

As per Article C5.6.3.2 of the AASHTO Code, the negative moment design section may be taken at the face of the support. It is conservative, but acceptable, to take the design section to be the centerline of the support. This will result in potentially much higher design moments.

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

Estimate Negative Moment Reinforcement

There are several methods of estimating the required amount of reinforcement in a non-prestressed section given the applied moments. One such method, shown below, involves approximation of a concrete tensile stress in the section, which is then used to determine how much reinforcement is required to resist that stress.

The approximate tensile stress in the concrete may be estimated as follows:

$$R_n = \frac{M_u}{\phi b d_s^{\,2}}$$

In which:

R_n = approximated concrete tensile stress (ksi)

M_u = Strength I moment (k-in.)

b = bottom flange width of prestressed beam (in.)

d_s = approximate distance from bottom of beam to centroid of slab reinforcement (in.). Because the exact centroid of slab reinforcement is unknown prior to design, using the centroid of the slab concrete is used as an approximation.

$$= h_{beam} + t_{fillet} + \frac{t_{slab}}{2}$$

ϕ = resistance factor for flexural resistance of non-prestressed concrete (5.5.4.2). This is most commonly 0.9, but can reduce to as low as 0.75 for an over-reinforced section. As an initial estimate, 0.9 is used.

The reinforcement ratio required to resist this stress is then calculated using the following equation:

$$\rho = \frac{\alpha_1 f'_{c,beam}}{f_y}\left[1-\sqrt{1-\frac{2R_n}{\alpha_1 f'_{c,beam}}}\right]$$

In which:

α_1 = stress block factor, taken as 0.85 for concrete with compressive strength not exceeding 10 ksi (5.6.2.2)

$f'_{c,beam}$ = 28-day compressive strength of concrete beam (ksi). Note that this is not the compressive strength of the slab

f_y = yield strength of slab reinforcement (ksi)

R_n = approximate concrete tensile stress (ksi)

Finally, the area of steel A_s is calculated from the reinforcement ratio:

A_s = $\rho b d_s$, in which ρ, b, and d_s are as defined above.

Determine Trial Reinforcement Configuration

Once the required area of steel is determined, a reinforcement layout is configured. The following rules are observed when determining a reinforcement layout:

- The bridge will have longitudinal reinforcement at 12 in. centers in the top of the slab, running the full length of the bridge. These bars are #6 bars at a minimum and #8 bars as a maximum. The bars are placed under the top of top slab transverse reinforcement.
- Full-length bars on the bottom of the slab should be detailed to avoid the vertical reinforcement protruding from the beams.
- Additional longitudinal bars are placed between the full-length bars. The spacing is therefore at 6 in. centers.
- The maximum bar size for the additional bars is #9.
- Additional bars should be placed in both the top of slab and bottom of slab.
- Ideally, 2/3 of the total reinforcement area should be in the top of slab and 1/3 of the total reinforcement area in the bottom of slab. This is not a rigid rule, but rather is a guideline to prevent either face from becoming over-reinforced.
- More than 1/3 of the total reinforcement is required to extend past the point of inflection (5.10.8.1.2c).
- No more than 50% of the bars may be cut off at the same location (5.10.8.1.2). Therefore, if more than 50% of the bars are required to be cut off, the reinforcement cutoffs are required to be staggered to meet this requirement.

Once a reinforcement layout is determined, the section can be designed.

Factored Flexural Resistance (5.6.3.2.1)

The flexural resistance of the negative moment region design section is calculated similarly to the positive moment region, except using the bottom of the prestressed beam as the compressive resistance component and the longitudinal slab reinforcement as the tensile resistance component. Due to the lack of prestressing in the slab, the equations simplify to those of a typical reinforced concrete section.

Because the bottom flange of a prestressed beam tapers, simplifications are made to approximate the beam shape into a rectangular shape. Figure 3 shows simplified compression block depths that are used as the bottom flange thicknesses in the equations below.

The design below is based upon Whitney stress block theory for nonprestressed sections. Some proprietary software gives designers the option for more refined models, such a strain compatibility model. While these more refined models are allowed as per the AASHTO Code, the resulting load rating generated by AASHTOWare uses Whitney stress block theory, and therefore may not be consistent with the load rating calculated by the designer's software.

$a = \beta_1 c$

Where:

c, neutral axis in bottom flange of beam

$$= \frac{A_s f_y}{\alpha_1 f'_{c,beam} \beta_1 b_{bf}} \qquad \text{(Eq. 5.6.3.1.1-4)}$$

c, neutral axis in web of beam

$$= \frac{A_s f_y - \alpha_1 f'_{c,beam} \left(b_{bf} - b_w\right) t_{bf}}{\alpha_1 f'_{c,beam} \beta_1 b_{bf}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

In these equations:

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

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

β_1 = stress block factor

= $0.65 \le 0.85 - 0.05\left(f'_c - 4.0\right) \le 0.85$ (5.6.2.2)

A_s = total area of longitudinal slab reinforcement (in.^{2})

f_y = yield strength of slab reinforcement (ksi)

α_1 = stress block factor

= 0.85 for design compressive strength not exceeding 10 ksi (5.6.2.2)

$f'_{c,beam}$ = specified 28 day compressive strength of precast prestressed concrete beam concrete (ksi)

b_{bf} = bottom flange width of precast prestressed concrete beam (in.)

b_w = web width of precast prestressed concrete beam (in.)

t_{bf} = simplified bottom flange depth, taken from Figure 3 (in.)

$$M_r = \phi M_n \quad \text{(Eq. 5.6.3.2.1-1)}$$

Where:

M_n, neutral axis in bottom flange of beam

$$= A_s f_y\left(d_s - \frac{a}{2}\right) \quad \text{(Eq. 5.6.3.2.2-1)}$$

M_n, neutral axis in web of beam

$$= A_s f_y\left(d_s - \frac{a}{2}\right) + \alpha_1 f'_c\left(b_{bf} - b_w\right)t_{bf}\left(\frac{a}{2} - \frac{t_{bf}}{2}\right) \quad \text{(Eq. 5.6.3.2.2-1)}$$

In which:

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

M_n = nominal flexural resistance (kip-in.)

ϕ = resistance factor, as calculated using Eq. 5.5.4.2.1-1. The resistance factor ϕ is dependent upon the amount of strain in the reinforcement. For tension-controlled sections, the resistance factor ϕ is 0.9.

$$\phi = 0.75 \le \phi = 0.75 + \frac{0.15\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \le 0.9 \quad \text{(Eq. 5.5.4.2-2)}$$

ε_t = net tensile strain in the extreme tension steel at nominal resistance, calculated using the procedure found in Article C5.6.2.1

$$= \frac{0.003\left(h_{beam} + t_{fillet} + t_{slab} - 2.25\text{ in. clear} - 0.625\text{ in. bar} - 0.5(d_{bar}) - c\right)}{c}$$

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

ε_{cl} = compression-controlled strain limit, taken as 0.002 for prestressed concrete sections (5.6.2.1)

ε_{tl} = tension-controlled strain limit, taken as 0.005 for prestressed concrete sections (5.6.2.1)

Minimum Reinforcement (5.6.3.3)

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.

$M_r \geq M_{cr}$ (5.6.3.3)

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

$M_{cr} = \gamma_3\gamma_1 S'_{ts} f_r$ (Eq. 5.6.3.3-1)

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

Where:

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

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

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. For locations where the neutral axis is in the bottom flange, traditional working stress formulas for stress calculation can be used to calculate deck stresses and cracked section modulus:

f_s = reinforcement stress in deck (ksi)

$$= \frac{M_s}{A_s jd}$$

Where:

M_s = applied moment (k-in.)

A_s = area of reinforcement (in.^{2})

$$j = 1 - \frac{k}{3}$$

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$$

$$\rho = \frac{A_s}{bd}$$

b = bottom flange width (in.)

d = distance from bottom of bottom flange to centroid of reinforcement in slab (in.)

$$n = \frac{E_s}{E_{c,beam}}$$

S_{bc} = cracked section modulus to bottom of beam (in.^{3})

$$= \frac{I_{cr}}{c_s}$$

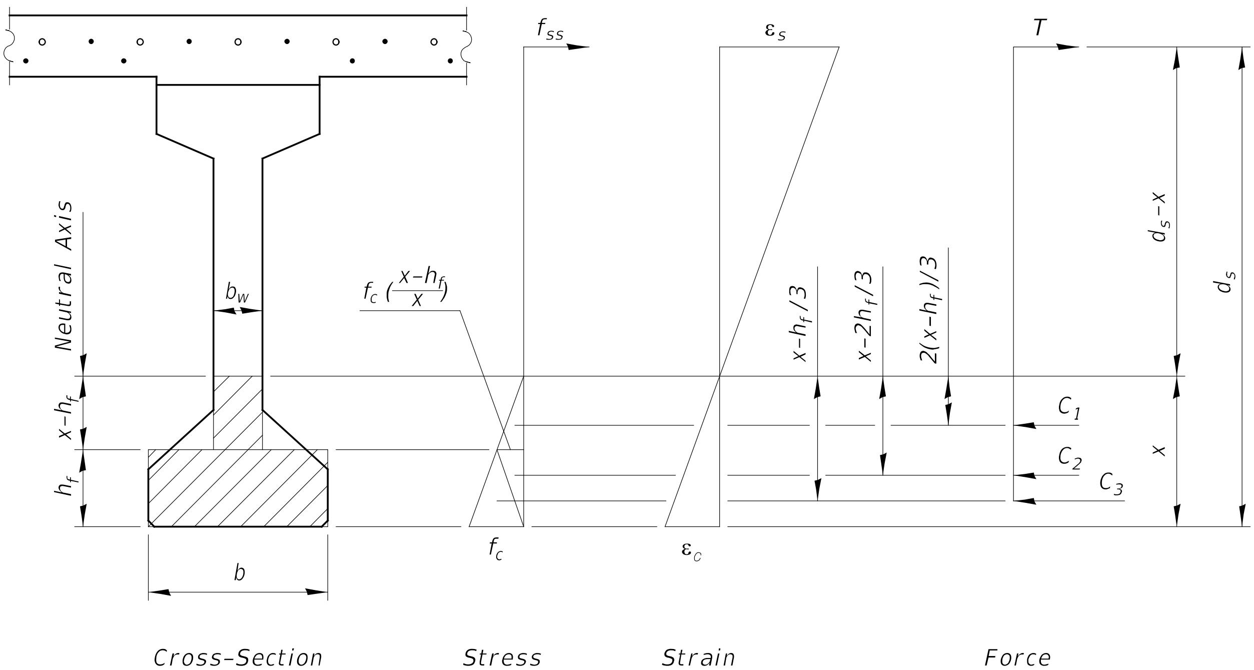
Where:

I_{cr} = cracked moment of inertia of composite section (in.^{4})

$$= \frac{bc_s^{\ 3}}{3} + nA_s\left(d - c_s\right)^2$$

$$c_s = n\rho d\left(\sqrt{1+\frac{2}{n\rho}} - 1\right)$$

For locations where the neutral axis is in the web, T-section behavior is observed, and the section modulus must be calculated using the procedure below. See Article 5.6.1 and Figure 4 below.



STRESS STRAIN DIAGRAM AT PIER

Figure 4

Determine expression for f_{ss}:

$f_{ss} = \varepsilon_s E_s$

$f_c = \varepsilon_c E_c$

$$\therefore\ \varepsilon_c = \frac{f_c}{E_c}$$

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

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

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

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

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

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

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

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

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> <u>(5.6.7)</u>

To control cracking, reinforcement spacing, s, shall meet the following requirement:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(Eq. 5.6.7-1)}$$

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 slab) (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 in slab, measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement in the slab at the service limit state (ksi)

h = overall thickness or depth of the component (equals total depth of beam and slab) (in.)

Fatigue of Reinforcement (5.5.3.2)

Reinforcement fatigue shall be controlled using the following procedure:

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

In which:

$(\Delta F)_{TH} = 26 - \frac{22 f_{min}}{f_y}$ (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
= 1.75

Service Limit State Stresses (5.9.2.3.2)

The compressive stress in the bottom flange is checked a transfer length from the end of the beam near the pier. The transfer length is 60 strand diameters as defined in Article 5.11.4.1.

The beam compressive stresses are checked for the same two Service limits and one Fatigue in the negative moment region as in the positive moment region.

Beam tensile stresses are not required to be checked in the top of the beam in the negative moment region for Service III limit state. This is because the deck is designed to control cracking, preventing cracking from reaching the beam section.

Designers should note that some design software checks Service III tensile stresses in tops of PPC beams in negative moment regions, and may return results showing excessive tensile stresses. These stresses are not considered to be problematic in design.

Service Stress Limits for Concrete after Losses (5.9.2.3.2)

Compression (For Service I load combination):

$0.60\phi_w f'_c$ (a) (Table 5.9.2.3.2a-1)

$0.45f'_c$ (b) (Table 5.9.2.3.2a-1)

Service stresses are calculated from the following equations:

@ Transfer point from pier:

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

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

Where:

F_S = total prestressing force after all losses (kips)

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

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

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

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

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

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

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 Limit State Stresses (5.5.3.1)

Fatigue limit state compressive stresses are also checked at the transfer length of the beam. Fatigue I tensile stresses are not checked because, similarly to the Service limit state stress checks, the beam is already conservatively checked using a cracked section.

$f_b < 0.4f'_c$ (5.5.3.1)

Fatigue stress is calculated from the following equation:

@ Transfer point from pier:

$$f_t = 0.5\left[\frac{F_S}{A} + \frac{F_S e}{S_{bc}} - \frac{(M_{DC1} + M_{DW1})}{S_{bc}} + \frac{(M_{DC2} + M_{DW2})}{S_{bc}}\right] + 1.75\frac{M_{FL+IM}}{S_{bc}}$$

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-in.)

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

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

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

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

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

<u>Estimate Area Required at Cutoff Point and Determine Cutoff Point</u>

The amount of reinforcement calculated at the negative moment design section is excessive in other areas of the bridge where there are lesser negative moments. It is therefore desirable to reduce the reinforcement accordingly by cutting off the additional bars. To determine cutoff points for reinforcement, the following AASHTO requirements are considered:

- No more than 50% of reinforcement may be terminated at one section (5.10.8.1.2a)
- At least 1/3 of reinforcement is extended beyond the point of contraflexure (5.10.8.1.2c)

Additionally, the Department requires that all reinforcement extend no less than a distance of 0.2L from either side of the pier (where L is the length of span under consideration). See Fig. 3.2.4-7 of the Bridge Manual.

Meeting these requirements may result in multiple cutoff locations for bridges, depending upon the layout of the slab reinforcement.

The theoretical cutoff point for the longitudinal reinforcement is located where the resisting moment capacity of the continuing reinforcement is greater than or equal to the applied moment at that location. This location can then be compared to the point of contraflexure and 20% of the span length, with a final cutoff location chosen that will have the simplest reinforcement detailing.

Designers should note that reinforcement may be required to extend further than required for strength in order to meet fatigue requirements. Designers should also note that fatigue of reinforcement need only be checked in high-stress regions, which are defined as 1/3 of the span on each side of the section of maxium moment. Therefore, reinforcement should never need to extend beyond 1/3 of the span length unless required for flexural capacity.

As per 5.10.8.1.2c, reinforcement is required to extend beyond the point where it is no longer needed by not less than the greatest of the effective depth of the member, 12.0 times the bar diameter, or 1/16 of the clear span. This may exceed the development length of the bar.

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, which are in turn based upon time-dependent camber equations in ACI, for the time at erection and have been incorporated into the equations shown below.

<u>Initial Resultant Camber</u>

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

In which:

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

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

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

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 (in.)

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/in.)

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.

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

In which:

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

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/in.)

L = span length (in.)

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)*

Final spacing of transverse reinforcement is checked at the critical sections for shear as well as at the tenth points along the beam or span. Final spacing for reinforcement is taken as the smallest spacing, enveloping following requirements:

Nominal Shear Resistance	(5.7.3.3)
Maximum Spacing of Transverse Reinforcement	(5.7.2.6)
Minimum Transverse Reinforcement	(5.7.2.5)
Interface Shear Transfer	(5.7.4)

In addition, longitudinal reinforcement in beams shall be checked near supports according to Article 5.7.3.5.

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.

Location of Critical Section (5.7.3.2)

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 face of the concrete diaphragm for integral abutments and fixed piers, and the edge of the bearing for abutments and piers with bearings.

Nominal Shear Resistance (5.7.3.3)

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

$$V_r = \phi V_n \qquad \text{(Eq. 5.7.2.1-1)}$$

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:

$$V_n = V_c + V_s + V_p \qquad \text{(Eq. 5.7.3.3-1)}$$

$$V_n = 0.25f'_c b_v d_v + V_p \qquad \text{(Eq. 5.7.3.3-2)}$$

Where:

V_c = Shear resistance due to concrete (k)

$$= 0.0316\beta\sqrt{f'_c}\,b_v d_v \qquad \text{(Eq. 5.7.3.3-3)}$$

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

$$= \frac{A_v f_y d_v \cot\Theta}{s} \qquad \text{(Eq. C5.7.3.3-1)}$$

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 5.7.3.3-1, 5.7.3.3-3, C5.7.3.3-1, and 5.7.2.1-1 for s and setting V_r equal to V_u gives a maximum spacing of:

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

As Equation 5.7.3.3-2 typically does not control the design, design for Equation 5.7.3.3-1 first, then check Equation 5.7.3.3-2 when a final design is reached.

<u>Effective Shear Depth d_v</u> <u>(5.7.2.8)</u>

The effective shear depth, d_v, is taken as:

d_v = distance between the resultants of the tensile and compressive forces due to flexure (in.)

$$= d_e - \frac{a}{2} \qquad (5.7.2.8)$$

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.

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. C5.9.4.3.2-1.

Ψ = Angle of harped strands from horizontal (degrees)

Shear Resistance Due to Concrete V_c (5.7.3.3)

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

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \quad \text{(Eq. 5.7.3.3-3)}$$

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 5.7.2.5 (Eq. 5.7.3.4.2-1). Assume that the requirements of Article 5.7.2.5 are satisfied, then check later in design.

Where:

ε_s = longitudinal strain (in./in.)

$$= \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{\left(E_sA_s + E_pA_{ps}\right)} \geq 0 \qquad \text{(Eq. 5.7.3.4.2-4)}$$

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. 5.7.3.4.2-1 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 5.7.3.3-1, 5.7.3.3-3, C5.7.3.3-1, and 5.7.2.1-1 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}). #5 bars are standard.

f_y = Yield strength of transverse reinforcement (ksi). 60 ksi should be assumed. This gives fabricators the ability to use either bar reinforcement or welded wire reinforcement.

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 (5.7.2.6)

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

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

$$s_{max} = 0.8d_v \leq 24.0 \text{ in.} \quad \text{(Eq. 5.7.2.6-1)}$$

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

$s_{max} = 0.4d_v \leq 12.0$ in. (Eq. 5.7.2.6-2)

Where:

f'_c = Compressive strength of concrete (ksi)

v_u = Shear stress on concrete (ksi)

$$= \frac{|V_u - \phi V_p|}{\phi b_v d_v} \quad \text{(Eq. 5.7.2.8-1)}$$

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 (5.7.2.5)

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}). #5 bars are standard.

f_y = Yield strength of transverse reinforcement (ksi)

f'_c = Compressive strength of concrete (ksi)

b_v = Width of web of member (in.)

Interface Shear Transfer Reinforcement (5.7.4)

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}, is taken as:

$V_{ui} = \phi V_{ni}$ (Eq. 5.7.4.3-1)

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:

$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c)$ (Eq. 5.7.4.3-3)

But not greater than the lesser of:

$V_{ni} \leq K_1 f'_c A_{cv}$ (Eq. 5.7.4.3-4)

$V_{ni} \leq K_2 A_{cv}$ (Eq. 5.7.4.3-5)

Where:

c = Cohesion factor specified in Article 5.7.4.3 (ksi). IDOT PPC beams have roughened top flanges to an amplitude of 0.25 in.

A_{cv} = Area of concrete considered to be engaged in interface shear transfer
= $b_{vi}L_{vi}$, (Eq. 5.7.4.3-6)

Where:

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

L_{vi} = 12 inch length along beam

μ = Friction factor specified in Article 5.7.4.3, taken as 0.28 for IDOT precast prestressed 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 5.7.4.3, taken as 0.3 for IDOT precast prestressed beams

K_2 = Limiting interface shear resistance specified in Article 5.8.4.3 (ksi), taken as 1.8 ksi for IDOT precast prestressed beams

Substituting $\frac{12A_{vf}}{s}$ for A_{vf} and solving Equations 5.7.4.3-1 and 5.7.4.3-3 for s and setting V_{ri} equal to V_{ui} gives the following equation for maximum spacing:

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

Where:

$$V_{ui} = v_{ui} A_{cv} \qquad \text{(Eq. 5.7.4.5-2)}$$

Where:

$$v_{ui} = \frac{V_u}{b_{vi} d_v} \qquad \text{(Eq. 5.7.4.5-1)}$$

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 5.7.4.3-4 and 5.7.4.3-5 are required to be checked:

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

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

If either of these two requirements is not satisfied, the lesser value of $K_1f'_cA_{cv}$ or K_2A_{cv} should be substituted for $\frac{V_{ui}}{\phi}$ when calculating the required spacing for interface shear.

Per Article 5.7.4.2, the minimum required area of interface shear reinforcement is:

$$A_{vf} = \frac{0.05A_{cv}}{f_y} \qquad \text{(Eq. 5.7.4.2-1)}$$

But need not exceed the amount needed to resist $\frac{1.33V_{ui}}{\phi}$ as determined using Eq. 5.7.4.3-3. Equation 5.7.4.2-1 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.

<u>Longitudinal Reinforcement</u> <u>(5.7.3.5)</u>

AASHTO 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:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{|V_u|}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad \text{(Eq. 5.7.3.5-2)}$$

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)

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

Piers

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 Θ 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:

$$\frac{d_v(1+\cot\theta)+x_{face}}{L} < 0.2 \qquad \text{(C5.7.3.5, modified)}$$

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:

$$A_s f_y + A_{ps} f_{ps} \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta \qquad \text{(Eq. 5.7.3.5-1)}$$

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 = resistance factor, as calculated using Eq. 5.5.4.2.1-1. The resistance factor ϕ is dependent upon the amount of strain in the reinforcement. For tension-controlled sections, the resistance factor ϕ is 0.9.

ϕ = $0.75 \le \phi = 0.75 + \dfrac{0.15\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \le 0.9$ (Eq. 5.5.4.2-1)

ε_t = net tensile strain in the extreme tension steel at nominal resistance, calculated using the procedure found in Article C5.6.2.1

$$= \frac{0.003\left(h_{beam} + t_{fillet} + t_{slab} - 2.5\text{ in. clear} - 0.5(0.625\text{ in. bar}) - c\right)}{c}$$

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

ε_{cl} = compression-controlled strain limit, taken as 0.002 for prestressed concrete sections (5.6.2.1)

ε_{tl} = tension-controlled strain limit, taken as 0.005 for prestressed concrete sections (5.6.2.1)

$|V_u|$ = 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 $\dfrac{|V_u|}{\phi_v}$

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. 5.7.3.3-2

$V_n = 0.25f'_c b_v d_v + V_p$ (Eq. 5.7.3.3-2)

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

Two-span, symmetric bridge with fixed pier and integral abutments. The Type, Size, and Location (TSL) Report shows a distance from the back of abutment to centerline of pier is 161 ft. - 1 in. The TSL Report also shows a 72” PPC IL-Beam with 7 ft. beam spacing and 3 ft. overhangs, 6 beams, 8 in. deck thickness, 44 in. constant-slope parapets, zero skew, 50 psf future wearing surface, and HL-93 loading.

The design code is the AASHTO LRFD Bridge Design Specifications, 8^{th} Ed.

Section Properties

Beam Selection

The planning charts for multi-span PPC bridges found in All Bridge Designers Memorandum 15.2 show that both beams (IL72-2438 and IL72-3838) can be used for the beam spacing and span length shown on the TSL.

To save on material, the smaller beam section (IL72-2438) is chosen.

Beam section = IL72-2438

Non-Composite Beam Section Properties (Fig. 1, ABD 15.2)

From Fig. 1, ABD 15.2, the following section properties are given for a non-composite IL72-2438 beam:

Area of beam, A_{beam} = 980.0 in.^{2}
Strong-axis moment of inertia, I_{xx} = 624180 in.^{4}
Section modulus to bottom of beam, S_b = 21237.8 in.^{3}
Section modulus to top of beam, S_t = 14648.6 in.^{3}
Distance from bottom of beam to centroid of beam, C_b = 29.39 in.
Distance from top of beam to centroid of beam, C_b = 42.61 in.

Moduli of Elasticity

E_p = 28500 ksi (5.4.4.2)

E_s = 29000 ksi (5.4.3.2)

$E_{ci} = 120{,}000 K_1 w_{ci}^2 f'^{0.33}_{ci}$ (Eq. 5.4.2.4-1)

$E_c = 120{,}000 K_1 w_c^2 f'^{0.33}_{c}$

Where:

$f'_{c,deck}$ = 4 ksi

$f'_{ci,beam}$ = 6.5 ksi

$f'_{c,beam}$ = 8.5 ksi

w_c = unit weight of concrete (kcf)

= 0.145 kcf for concrete with $f'_c \leq 5.0$ ksi (Table 3.5.1-1)

= 0.140 + 0.001f'_c kcf for concrete with 5.0 ksi < $f'_c \leq 15.0$ ksi (Table 3.5.1-1)

$w_{c,slab}$ = 0.145 kcf for 4.0 ksi concrete deck

$w_{ci,beam}$ = 0.140 + 0.001(6.5 ksi) = 0.1465 kcf for beam with f'_{ci} = 6.5 ksi

$w_{c,beam}$ = 0.140 + 0.001(8.5 ksi) = 0.1485 kcf for beam with f'_c = 8.5 ksi

$$E_{c,deck} = 120{,}000(1.0)\left(0.145\ \text{k / ft.}^3\right)^2(4\ \text{ksi})^{0.33} = 3987\ \text{ksi}$$

$$E_{ci,beam} = 120{,}000(1.0)\left(0.1465\ \text{k / ft.}^3\right)^2(6.5\ \text{ksi})^{0.33} = 4777\ \text{ksi}$$

$$E_{c,beam} = 120{,}000(1.0)\left(0.1485\ \text{k / ft.}^3\right)^2(8.5\ \text{ksi})^{0.33} = 5362\ \text{ksi}$$

Modular Ratio

$$n = \frac{E_{c,deck}}{E_{c,beam}} = \frac{3987\ \text{ksi}}{5362\ \text{ksi}} = 0.74$$

Composite Beam Section Properties

Effective Flange Width **(4.6.2.6)**

Dist. C. to C. of beams = 7 ft.(12 in./ft.) = 84 in.

Fillets

For the calculation of composite section properties, no fillet will be included in the eccentricity calculations. This conservatively assumes that the beams will be over-cambered to the point where the 0.5 in. minimum fillet detailed on the plans is not present.

Positive Moment Composite Design Section

Composite section neutral axis height, y'_b:

Area of beam, A_{beam} = 980.0 in.^{2} (See above)

Distance from bottom of beam to centroid of beam, C_b = 29.39 in. (See above)

Transformed area of slab, A_{slab} = nt_{slab}(eff. flange width)
= 0.74(8 in.)(84 in.)
= 497.3 in.^{2}

Distance from bottom of beam to centroid of slab, y'_{slab}
= 72 in. beam + 0 in. fillet + 0.5(8 in. slab)
= 76 in.

	A	y	Ay
Slab	497.3 in.^{2}	76 in.	37793.3 in.^{3}
Beam	980.0 in.^{2}	29.39 in.	28802.2 in.^{3}
Total	1477.3 in.^{2}		66595.5 in.^{3}

$$y'_b = \frac{66595.5 \text{ in.}^3}{1477.3 \text{ in.}^2} = 45.08 \text{ in.}$$

Distance from composite neutral axis to top of beam y'_t:

$$y'_t = h_{beam} - y'_b$$
$$= 72 \text{ in.} - 45.08 \text{ in.} = 26.92 \text{ in.}$$

Composite beam section moduli S'_b and S'_t:

$$I_{o,slab} = \frac{1}{12}(n)(\text{Eff. flange width})(t_{slab})^3$$
$$= \frac{1}{12}(0.74)(84 \text{ in.})(8 \text{ in.})^3$$
$$= 2652.2 \text{ in.}^4$$

$$A_{slab} = 497.3 \text{ in.}^2$$

d_{slab} = distance from centroid of slab to centroid of composite section (in.)

$$= y'_{slab} - y'_b$$
$$= 76 \text{ in.} - 45.08 \text{ in.}$$
$$= 30.92 \text{ in.}$$

$I_{o,beam}$ = 624180 in.^{4} (see above)

A_{beam} = 980 in.^{2} (see above)

d_{beam} = distance from centroid of beam to centroid of composite section (in.)

= $C_b - y'_b$

= 29.39 in. – 45.08 in.

= -15.69 in.

	I_o	A	d	Ad^2	I'
Slab	2652.2 in.^{4}	497.3 in.^{2}	30.92 in.	475443 in.^{4}	478075 in.^{4}
Beam	624180 in.^{4}	980.0 in.^{2}	-15.69 in.	241253 in.^{4}	865433 in.^{4}
				I' =	1343507 in.^{4}

$$S'_b = \frac{I'}{y'_b} = \frac{1343507 \text{ in.}^3}{45.08 \text{ in.}^2} = 29802.74 \text{ in.}^3$$

$$S'_t = \frac{I'}{y'_t} = \frac{1343507 \text{ in.}^3}{26.92 \text{ in.}^2} = 49907.41 \text{ in.}^3$$

Negative Moment Composite Design Section

Because the slab reinforcement is required to be known in order to calculate negative moment composite design section properties, those properties will be calculated in the Negative Moment Region Design section of the design guide.

Loading

Span Lengths of Noncomposite Beams and Composite Sections

Noncomposite beam span length = CL Brg. Abut. to CL Brg. Pier

= (161 ft. – 1 in.) – (1 ft. – 11 ½ in.) – (1 ft. – 1 ½ in.)

= 161.083 ft. – 1.958 ft. – 1.125 ft.

= 158.0 ft.

Composite section span length = CL Brg. Abut. To CL Pier

= (161 ft. – 1 in.) – (1 ft. – 11 ½ in.)

= 161.083 ft. – 1.958 ft.

= 159.125 ft.

Non-Composite (DC1) Dead Loads and Composite (DC2, DW) Dead Loads

DC1: Beam = 1.021 k / ft. (Fig. 1, ABD 15.2)

Fillet = $w_c(t_{fillet})(b_{tf})$

$$= \left(\frac{0.150\text{ k}}{\text{ft.}^3}\right)(2\text{ in.})(24\text{ in.})\left(\frac{1\text{ ft.}^2}{144\text{ in.}^2}\right) = 0.050\text{ k / ft.}$$

Slab = w_c(Eff. flange width)(t_{slab})

$$= \left(\frac{0.150\text{ k}}{\text{ft.}^3}\right)(7\text{ ft.})(8\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) = 0.700\text{ k / ft.}$$

Total = = 1.771 k / ft.

DC2: 44 in. Constant-Slope Parapet = 0.38 ft.^{3} / ft. (ABD 19.1)

Weight per foot = (0.38 ft.^{3} / ft.) (0.150 k /ft^3) = 0.570 k / ft.

$$\text{Parapet} = \left(\frac{0.570\text{ k}}{\text{ft. / parapet}}\right)\left(\frac{2\text{ parapets}}{6\text{ beams}}\right) = 0.190\text{ k / ft.}$$

DW: FWS
$$= \left(\frac{0.050\text{ k}}{\text{ft.}^2}\right)(7\text{ ft.}) = 0.350\text{ k / ft.}$$

Live Load Distribution Factors (4.6.2.2)

Longitudinal Stiffness Parameter K_g **(4.6.2.2.1)**

$$K_g = n\left(I + Ae_g^2\right) \quad \text{(Eq. 4.6.2.2.1-1)}$$

Where:

n = modular ratio of beam to deck. Note that this is the reciporocal of “n” calculated above.

$$= \frac{E_{c,beam}}{E_{c,deck}} = \frac{5362\text{ ksi}}{3987\text{ ksi}} = 1.34$$

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

= 624180 in.^{4}

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

= 980 in.^{2}

e_g = distance from centroid of deck to centroid of beam

= y'_t + fillet + $0.5t_{slab}$

= 42.61 in. + 0 in. + 0.5(8 in.)

= 46.61 in.

$$K_g = 1.34\left(624180\text{ in.}^4 + \left(980\text{ in.}^2\right)\left(46.61\text{ in.}\right)^2\right)$$

$$= 3689318\text{ in.}^4$$

Live Load Distribution Factor, Moment, Interior Beams **(Table 4.6.2.2.2b-1)**

g_1 = single lane live load distribution factor

$$= 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

Where:

S = beam spacing (ft.)

= 7 ft.

L = composite span length (ft.)

= 159.125 ft.

t_s = slab thickness (in.)

= 8 in.

K_g = longitudinal stiffness parameter (in.^{4})

= 3689318 in.^{4}

$$g_1 = 0.06 + \left(\frac{7}{14}\right)^{0.4}\left(\frac{7}{159.125}\right)^{0.3}\left(\frac{3869318}{12.0(159.125)(8)^3}\right)^{0.1}$$

$$= 0.401$$

g_m = multiple lane live load distribution factor

$$= 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

$$= \quad 0.075+\left(\frac{7}{9.5}\right)^{0.6}\left(\frac{7}{159.125}\right)^{0.2}\left(\frac{3869318}{12.0(159.125)(8)^{3}}\right)^{0.1}$$

$$= \quad 0.587$$

$g_m > g_1$, therefore multiple lane loading controls, g = 0.587

$g_{fatigue}$ = live load distribution factor for fatigue limit states

= $\frac{g_1}{m}$, where m = 1.2 for single-lane loading (3.6.1.1.2)

$$= \quad \frac{0.401}{1.2}$$

$$= \quad 0.334$$

Live Load Distribution Factor, Shear and Reaction, Interior Beams

(Table 4.6.2.2.3a-1)

$$g_1 = 0.36+\frac{S}{25.0}$$

$$= 0.36+\frac{7}{25.0}$$

$$= 0.640$$

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

$$= 0.2+\left(\frac{7}{12}\right)-\left(\frac{7}{35}\right)^{2.0}$$

$$= 0.743$$

$g_m > g_1$, therefore multiple lane loading controls, g = 0.743

Skew correction = $1+0.2\left(\frac{12.0Lt_s^{3}}{K_g}\right)\tan(\theta)$ (Table 4.6.2.2.3c-1)

$$= 1 + 0.2\left(\frac{12.0(159.125)(8)^3}{3869318}\right)\tan(0)$$

$$= 1.0$$

Live Load Distribution Factor, Deflection

$$g_{defl} = m\left(\frac{N_L}{N_b}\right)$$

Where:

m = 0.85 for three lanes (Table 3.6.1.1.2-1)

N_L = number of lanes

= integer part of the ratio w/12.0, where w is the clear roadway width in feet between barriers (3.6.1.1.1)

$$= \left((5\text{ bm. spc.})\left(\frac{7\text{ ft.}}{\text{spc.}}\right) + 2\text{ overhangs}\left(\frac{3\text{ ft.}}{\text{overhang}}\right) - 2\text{ parapets}\left(\frac{1.417\text{ ft.}}{\text{parapet}}\right)\right)/12.0$$

= 38.167 ft. / 12.0

= 3.2 lanes, therefore use 3 lanes.

N_b = number of beams

= 6 beams

$$g_{defl} = 0.85\left(\frac{3}{6}\right)$$

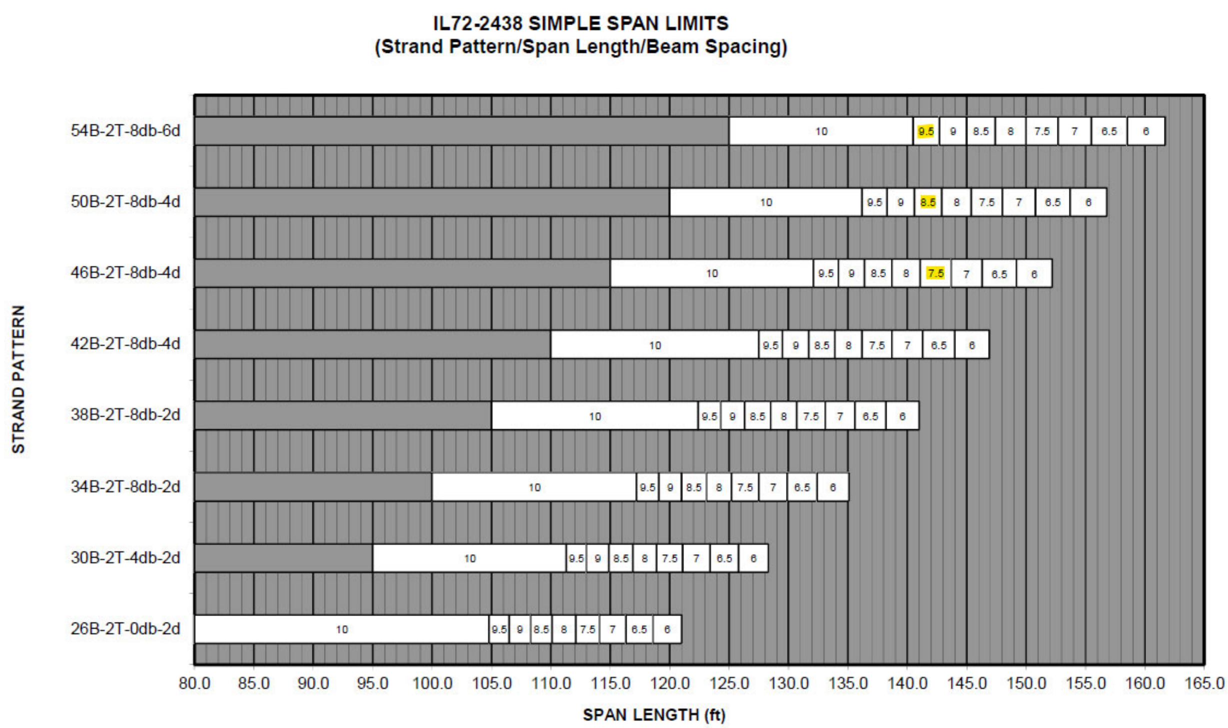
$$= 0.425$$

Moment and Shear Envelopes

Moment and shear envelopes have been generated using proprietary software. These envelopes are given in Appendix A.

Standard Strand Patterns

From Fig. 15 of ABD 15.2, strand patterns 46B-2T-8db-4d, 50B-2T-8db-4d, and 54B-2T-8db-6d all will work as trial strand patterns for a span length of 0.9(158 ft.) = 142.2 ft. and a beam spacing of 7 ft. or greater:



However, initial checks show that stand pattern 46B-2T-8db-4d may not be adequate in tension at the point of maximum moment. For this reason, choose strand pattern 50B-2T-8db-4d.

Strand Pattern = 50B-2T-8db-4d

Meaning:

50B	=	50 strands at the bottom of the beam at midspan
2T	=	2 strands running full length along the top of the beam
8db	=	8 debonded strands
4d	=	4 draped strands

The number of strands per row, draped strand locations, and debonded strand rows and locations are found in Figure 26 of ABD 15.2:

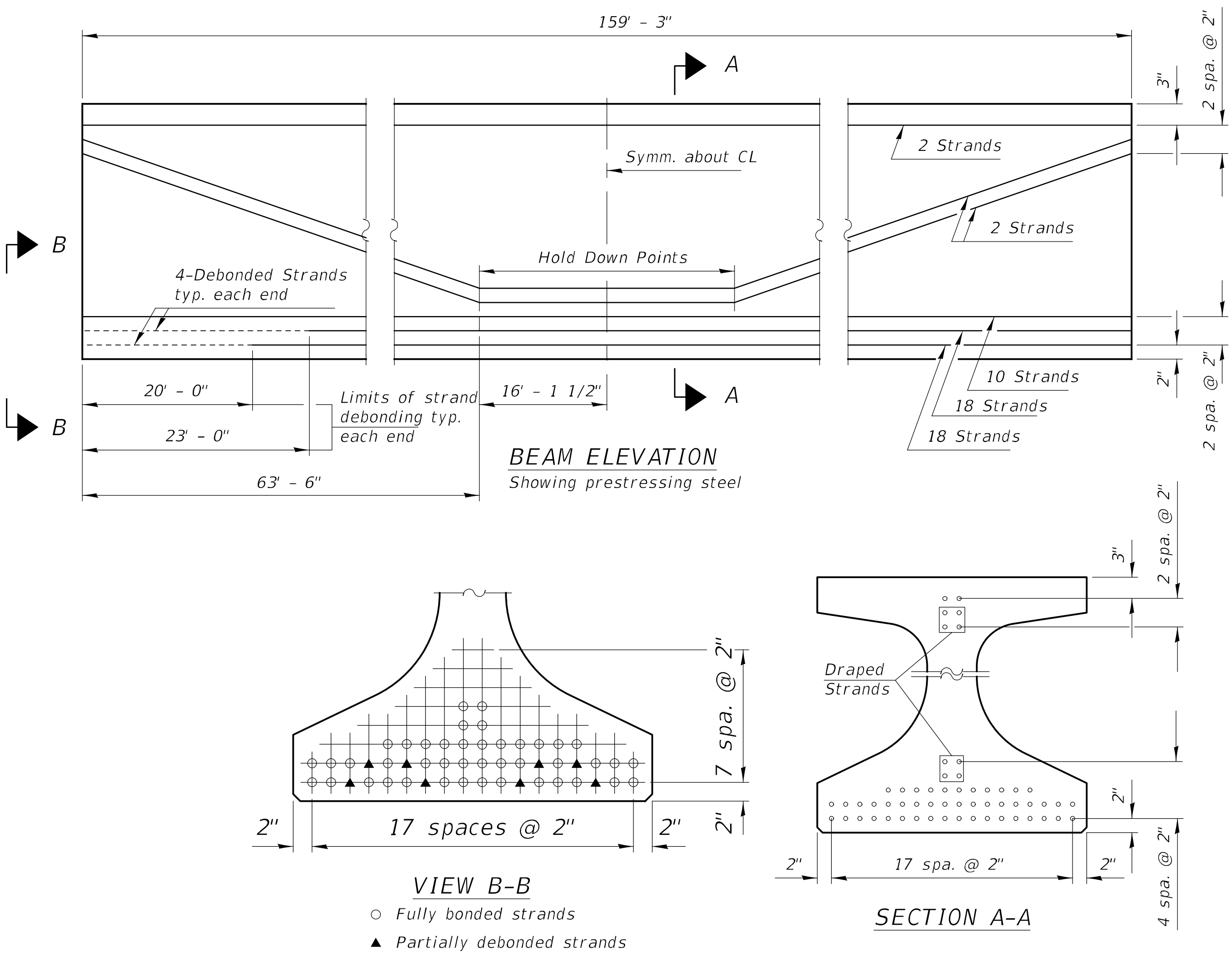
IL72-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																			Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)												
26B-2T-0db-2d	Ctr.	18			8												2					80
	End	18			6												2	2				
30B-2T-4db-2d	Ctr.	18			12												2					95
	End	18	4	216	10												2	2				
34B-2T-8db-2d	Ctr.	18			16												2					100
	End	18	4	216	14	4	252										2	2				
38B-2T-8db-2d	Ctr.	18			16			2			2						2					105
	End	18	4	216	16	4	252	2									2	2				
42B-2T-8db-4d	Ctr.	18			18			6									2					110
	End	18	4	216	16	4	252	4									2	2	2			
46B-2T-8db-4d	Ctr.	18			18			10									2					115
	End	18	4	240	16	4	276	8									2	2	2			
50B-2T-8db-4d	Ctr.	18			18			10			2	2					2					120
	End	18	4	240	18	4	276	10									2	2	2			
54B-2T-8db-6d	Ctr.	18			18			16			2						2					125
	End	18	4	240	16	4	276	14									2	2	2	2		

This gives the following strand locations:

- 18 strands in row 1B, all straight strands, with 4 strands debonded for the first 240 in.
- 18 strands in row 2B, all straight strands, with 4 strands debonded for the first 276 in.
- 10 strands in row 3B, all straight strands, none debonded
- 2 strands in row 4B, harped to row 3T at the end of the beam
- 2 strands in row 5B, harped to row 2T at the end of the beam
- 2 strands in row 1T, running straight along the top of the beam for the entire beam length

Detail of Strand Pattern on Plans



Prestress Losses *(5.9.3)*

Total Loss of Prestress (5.9.3.1)

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.3.1-1)

Instantaneous Losses (5.9.3.2)

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ (Eq. 5.9.3.2.3a-1)

Where:

E_p = 28500 ksi (5.4.4.2)

E_{ci} = 4777 ksi

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

Where:

F_t = assume F_t equals 90 percent of F_i for first iteration:

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

A_{ps} = 0.217 in.^{2} / strand * 52 strands

= 11.284 in.^{2}

f_{pbt} = 202.3 ksi

F_i = (11.284 in.^{2})(202.3 ksi) = 2283 kips

F_t = 0.9(F_i)

= 0.9(2283 k)

= 2055 kips

A = 980 in.^{2}

e = strand pattern eccentricity at point of maximum moment

= C_b – height of centroid of strand group

Determine height of centroid of strand group at point of maximum moment:

At the center of the beam, strand pattern 50B-2T-8db-4d has the following configuration:

Row	Row Height m (in.)	# of Strands in Row n	m * n
1B	2	18	36
2B	4	18	72
3B	6	10	60
4B	8	2	16
5B	10	2	20
1T	69	2	138
Total:		52	342

Height of centroid = 342 / 52

= 6.58 in. from bottom of beam

$$e = 29.39 \text{ in.} - 6.58 \text{ in.}$$

$$= 22.81 \text{ in.}$$

$$M_b = \frac{(1.021\text{k / ft.})(159.25\text{ ft.})^2}{8} = 3237\text{ k-ft.}$$

$$f_{cgp} = \frac{2055\text{ k}}{980\text{ in.}^2} + \frac{(2055\text{ k})(22.81\text{ in.})^2}{624180\text{ in.}^4} - \frac{(3237\text{ k} - \text{ft.})(12\text{ in./ ft.})(22.81\text{ in.})}{624180\text{ in.}^4}$$

$$= 2.39\text{ ksi}$$

$$\Delta f_{pES} = \frac{28500\text{ ksi}}{4777\text{ ksi}}(2.39\text{ ksi}) = 14.26\text{ ksi}$$

Check Assumption that $F_t = 0.9F_i$:

$$\frac{f_{pbt} - \Delta f_{pES}}{f_{pbt}} = \frac{202.3\text{ ksi} - 14.26\text{ ksi}}{202.3\text{ ksi}} = 0.93 > 0.90$$

Assume F_t equals 93 percent of F_i for second iteration:

$$F_t = 0.93(2283\text{ kips}) = 2123\text{ kips}$$

$$f_{cgp} = \frac{2123\text{ k}}{980\text{ in.}^2} + \frac{(2123\text{ k})(22.81\text{ in.})^2}{624180\text{ in.}^4} - \frac{(3237\text{ k}-\text{ft.})(12\text{ in./ft.})(22.81\text{ in.})}{624180\text{ in.}^4}$$

$$= 2.52\text{ ksi}$$

$$\Delta f_{pES} = \frac{28500\text{ ksi}}{4777\text{ ksi}}(2.52\text{ ksi}) = 15.03\text{ ksi}$$

Check Assumption that $F_t = 0.93F_i$:

$$\frac{f_{pbt} - \Delta f_{pES}}{f_{pbt}} = \frac{202.3\text{ ksi} - 15.03\text{ ksi}}{202.3\text{ ksi}} = 0.93 \quad \text{OK}$$

Approximate Estimate of Time Dependent Losses (5.9.3.3)

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(Eq. 5.9.3.3-1)}$$

In which:

f_{pbt} = 202.3 ksi for 0.6 in. diameter low-relaxation strands

A_{ps} = 11.284 in.^{2}

A = 980 in.^{2}

γ_h = 1.7-0.01H (Eq. 5.9.3.3-2)

= 1.7-0.01(70)

= 1.0

$$\gamma_{st} = \frac{5}{1+f'_{ci}} \qquad \text{(Eq. 5.9.3.3-3)}$$

$$= \frac{5}{1+6.5\text{ ksi}}$$

= 0.667

Δf_{pR} = 2.4 ksi

$$\Delta f_{pLT} = 10.0\frac{(202.3\text{ ksi})(11.284\text{ in.}^2)}{(980\text{ in.}^2)}(1.0)(0.667) + 12.0(1.0)(0.667) + 2.4\text{ ksi}$$

$= 25.94$ ksi

<u>Total Loss of Prestress</u> (5.9.3.1)

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eq. 5.9.3.1-1)

$= \Delta f_{pES} + \Delta f_{pLT} = 15.03 \text{ ksi} + 25.94 \text{ ksi} = 40.97 \text{ ksi}$

$\%\text{Loss} = \dfrac{40.97 \text{ ksi}}{202.3 \text{ ksi}} = 20.3\ \%$

$f_{pbt} - \Delta f_{pT} < 0.8 f_{py}$ (Table 5.9.2.2-1)

202.3 ksi – 40.97 ksi = 161.33 ksi

$0.8f_{py} = 0.8(0.9f_{pu}) = 0.8(0.9)(270 \text{ ksi}) = 194.4$ ksi

161.33 ksi < 194.4 ksi OK

Strand Development Lengths *(5.9.4.3.2)*

$$\ell_d = K\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b$$ (Eq. 5.9.4.3.2-1)

Where:

$K = 1.6$ for bonded strands (5.9.4.3.2)

$= 2$ for debonded strands (5.9.4.3.2)

$f_{pe} = 202.3 \text{ ksi} * (1 – 20.3 / 100)$

$= 161.23$ ksi

$d_b = 0.6$ in.

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ (Eq. 5.6.3.1.1-1)

$f_{pu} = 270$ ksi

$k = 0.28$

c = Distance from extreme compression fiber to neutral axis, calculated assuming any debonded strands are not present in the strand group at the location of development length (in.)

A_{ps} = 44 strands(0.217 in.^{2} / strand)

= 9.548 in.^{2}

f_{pu} = 270 ksi

α_1 = 0.85

$f'_{c,slab}$ = 4 ksi

β_1 = 0.85 for 4 ksi concrete

b_{slab} = 84 in.

b_{tf} = $\frac{1}{n} b_{tf}$

= $\frac{1}{0.74}(24 \text{ in.})$

= 32.43 in.

k = 0.28

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

Estimate that the development length is 12 ft. for a 72 in. beam. Determine height of centroid of strand group at a distance 12 ft. from the beam end.

At this location, there are four debonded strands in each of the first two rows, making the effective number of strands equal to 14. Rows 4B and 5B are harped, and the harping angle is 4.26 degrees for this beam depth, beam length, and strand pattern. Therefore, their row heights are calculated to be (144 in.)(tan 4.26 degrees) = 10.7 in. from their location at the end of the beam:

Row	Row Height m (in.)	# of Strands in Row n	m * n
1B	2	14	28
2B	4	14	56
3B	6	10	60
4B	54.3	2	108.6
5B	56.3	2	112.6
1T	69	2	138
Total:		44	503.2

Height of centroid = 503.2 / 44

= 11.4 in. from bottom of beam

d_p = 72 in. beam + 1.5 in. fillet + 8 in. slab – 11.4 in.

= 70.1 in.

check c, neutral axis in slab

$$c = \frac{A_{ps}f_{pu}}{\alpha_1 f'_{c,slab}\,\beta_1 b_{slab} + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-4)}$$

$$= \frac{(9.548\text{ in.}^2)(270\text{ ksi})}{(0.85)(4\text{ ksi})(0.85)(84\text{ in.}) + (0.28)(9.548\text{ in.}^2)\dfrac{(270\text{ ksi})}{(70.1\text{ in.})}}$$

= 10.2 in.

a = 0.85(10.2 in.) = 8.67 in. > 8 in. slab thickness, neutral axis likely in top flange

check c, neutral axis in top flange of beam

$$= \frac{A_{ps}f_{pu} - \alpha_1 f'_{c,slab}(b_{slab} - b_{tf})t_{slab}}{\alpha_1 f'_{c,slab}\,\beta_1 b_{tf} + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

$$= \frac{(9.548\text{ in.}^2)(270\text{ ksi}) - 0.85(4\text{ ksi})(84\text{ in.} - 32.43\text{ in.})(8\text{ in.})}{(0.85)(4\text{ ksi})(0.85)(32.43\text{ in.}) + (0.28)(9.548\text{ in.}^2)\dfrac{(270\text{ ksi})}{(70.1\text{ in.})}}$$

= 11.3 in.

a = 0.85(11.3 in.) = 9.61 in. > 8 in. slab thickness and < 14.03 in. slab + top flange thickness, therefore neutral axis in top flange

$$f_{ps} = (270 \text{ ksi})\left(1 - 0.28\frac{11.3 \text{ in.}}{70.1 \text{ in.}}\right)$$

= 257.8 ksi

$$\ell_d = 1.6\left(257.8 \text{ ksi} - \frac{2}{3}161.23 \text{ ksi}\right)(0.6 \text{ in.})$$

= 144.3 in. for bonded strands

Note that this value is close to the value of 12 ft. chosen to begin interation.

$$\ell_d = 2.0\left(257.8 \text{ ksi} - \frac{2}{3}161.23 \text{ ksi}\right)(0.6 \text{ in.})$$

= 180.4 in. for debonded strands

Strand Group Eccentricities

Because the calculation of strand group eccentricities is repetitive and tabulated, only one section location will be fully calculated in this portion of the design guide. Full calculations for the strand group eccentricities in all locations may be found in Appendix B.

The section at the "Debond Location 2 + Transfer" will be shown for calculations. At this location, there are sets of strands at different levels of development.

$$\text{Angle of Inclination of Harped Strands} = \arctan\left(\frac{\text{harped strand drop in height}}{0.4 * \text{beam length}}\right)$$

The harped strands are in two rows. At the ends of the beams, the harped strands are in rows 2T and 3T, with heights of 67 in. and 65 in. above the bottom of the beam, respectively. At the harping point, the harped strands are in rows 5 and 4, with heights of 10 in. and 8 in. above the bottom of the beam. Therefore:

Harped strand drop in height = 67 in. – 10 in. = 57 in.

$$0.4 * \text{beam length} = 0.4(159.25\text{ ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)$$

$$= 764.4\text{ in.}$$

$$\text{Angle of inclination } \Psi = \arctan\left(\frac{57\text{ in.}}{764.4\text{ in.}}\right)$$

$$= 4.26^{o}$$

The “Debond Location 2 + transfer” length occurs one transfer length (60 strand diameters) past the point of the debonding of the second group of debonded strands. This occurs at:

276 in. + 60(0.6 in.) = 312 in.

$$\text{Total drop for harped strands} = \text{distance to location} * \tan\Psi$$

$$= 312\text{ in.} * \tan 4.26^{o}$$

$$= 23.27\text{ in.}$$

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	23.27	41.73
5B	67	Y	23.27	43.73
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	100	14	4	71.84	2.8736				16.8736
2B	14	100	14				4	62.5	2.5	16.5
3B	10	100	10							10
4B	2	100	2							2
5B	2	100	2							2
1T	2	100	2							2
									Σ =	49.3736

A_{ps} = 0.217 in.^{2} / strand * 49.37 eff. strands = 10.71 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	16.8736	33.7472
2B	4	16.5	66
3B	6	10	60
4B	41.73	2	83.46
5B	43.73	2	87.46
1T	69	2	138
		Σ =	468.6672

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,\text{Dev.})\right)}{\sum \Sigma(N * \%\,\text{Dev.})} = 29.39\text{ in.} - \frac{468.67\text{ in.}}{49.37} = 19.90\text{ in.}$$

Temporary Stresses

Lifting Loop Locations

Beam length = center-to-center bearing distance + 2 * 0.625 ft. beyond CL bearing
= 158 ft. + 2(0.625 ft.)
= 159.25 ft.

Total beam weight= $(1.021\text{ k/ft.})(159.25\text{ ft.})$ = 162.59 kips

From Fig. 28, ABD 15.2:

- For IL72-2438 beams with lengths greater than 159 feet but less than or equal to 164 ft., place first lifting loop 6 ft. from end of beam
- For beam weights greater than 138500 lbs. and less than or equal to 166200 lbs., use 4 lifting loops each beam end, with each additional lifting loop placed 4 ft. further into the span
- For beam weights greater than 138500 lbs. and less than or equal to 166200 lbs., use 3 strands per lifting loop

The centroid of lifting loops is therefore 6 ft. + 1.5(4 ft.) = 12 ft. from the beam end

Temporary Stress Limits (5.9.2.3.1)

Compression:

$0.65f'_{ci} \quad = \quad 0.65(6.5\text{ ksi}) \quad = \quad 4.225\text{ ksi}$ (5.9.2.3.1a)

Tension:

$-0.24\sqrt{f'_{ci}} \quad = \quad -0.24\sqrt{6.5\text{ ksi}} \quad = \quad -0.612\text{ ksi}$ (5.9.2.3.1b)

Load Conditions for Temporary Stresses

Temporary stresses will be checked at the following locations:

- Centroid of lifting loops
- First debond location
- Second debond location/First debond + transfer location
- Second debond + transfer location
- Harping point

@ Centroid of Lifting Loops (144 in. from Beam End)

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

$w_b = 1.021\text{ k / ft.}$

L_c = distance from end of beam to centroid of lifing loop (ft.)

= 12 ft.

$$M_{bts} = -\frac{(1.021 \text{ k / ft.})(12 \text{ ft.})^2}{2} = -73.51 \text{ k-ft.}$$

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

A_{ps} = 9.52 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pES} = 15.03 ksi

$$= 9.52 \text{ in.}^2 (202.3 \text{ ksi} - 15.03 \text{ ksi}) = 1783 \text{ kips}$$

e = 17.97 in. (See App. B)

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

$$= \frac{1783 \text{ k}}{980 \text{ in.}^2} - \frac{(1783 \text{ k})(17.97 \text{ in.})}{14648.6 \text{ in.}^3} + \frac{(-73.51 \text{ k} - \text{ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{14648.6 \text{ in.}^3}$$

= -0.42 ksi > -0.612 ksi OK

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

$$= \frac{1783 \text{ k}}{980 \text{ in.}^2} + \frac{(1783 \text{ k})(17.97 \text{ in.})}{21237.8 \text{ in.}^3} - \frac{(-73.51 \text{ k} - \text{ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{21237.8 \text{ in.}^3}$$

= 3.37 ksi < 3.60 ksi OK

@ Debond Location 1 (240 in. from Beam End)

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

w_b = 1.021 k / ft.

L_c = 12 ft.

L_b = 159.25 ft.

x = distance from debond location 1 to lifting loop location

= (240 in. – 144 in.)(1 ft. / 12 in.)

= 8 ft.

$$M_{bts} = -\frac{(1.021\text{ k / ft.})(12\text{ ft.})^2}{2} + \frac{(1.021\text{ k / ft.})(8\text{ ft.})}{2}(159.25\text{ ft.} - 2(12\text{ ft.}) - 8\text{ ft.})$$

$= 446.18$ k-ft.

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

$A_{ps} = 9.55\text{ in.}^2$ (See App. B)

$f_{pbt} = 202.3$ ksi

$\Delta f_{pES} = 15.03$ ksi

$$F_t = 9.55\text{ in.}^2(202.3\text{ ksi} - 15.03\text{ ksi})$$

$= 1788$ kips

$e = 18.61$ in. (See App. B)

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

$$= \frac{1788\text{ k}}{980\text{ in.}^2} - \frac{(1788\text{ k})(18.61\text{ in.})}{14648.6\text{ in.}^3} + \frac{(446.18\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$= -0.08$ ksi > -0.612 ksi OK

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

$$= \frac{1788\text{ k}}{980\text{ in.}^2} + \frac{(1788\text{ k})(18.61\text{ in.})}{21237.8\text{ in.}^3} - \frac{(446.18\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$= 3.13$ ksi < 4.225 ksi OK

@ Debond Location 1 + Transfer / Debond Location 2 (276 in. from Beam End)

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

$w_b = 1.021$ k / ft.

$L_c = 12$ ft.

$L_b = 159.25$ ft.

$x =$ distance from (debond location 1 + transfer) to lifting loop location

$= (276\text{ in.} - 144\text{ in.})(1\text{ ft.} / 12\text{ in.})$

$= 11\text{ ft.}$

$$M_{bts} = -\frac{(1.021\text{k / ft.})(12\text{ ft.})^2}{2} + \frac{(1.021\text{k / ft.})(11\text{ ft.})}{2}(159.25\text{ ft.} - 2(12\text{ ft.}) - 11\text{ ft.})$$

$= 624.21\text{ k-ft.}$

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

$A_{ps} = 10.10\text{ in.}^2$ (See App. B)

$f_{pbt} = 202.3\text{ ksi}$

$\Delta f_{pES} = 15.03\text{ ksi}$

$F_t = 10.10\text{ in.}^2(202.3\text{ ksi} - 15.03\text{ ksi})$

$= 1891\text{ kips}$

$e = 19.31\text{ in.}$ (See App. B)

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

$$= \frac{1891\text{k}}{980\text{ in.}^2} - \frac{(1891\text{k})(19.31\text{in.})}{14648.6\text{ in.}^3} + \frac{(624.21\text{k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$= -0.05\text{ ksi}$ $> -0.612\text{ ksi}$ OK

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

$$= \frac{1891\text{k}}{980\text{ in.}^2} + \frac{(1891\text{k})(19.31\text{in.})}{21237.8\text{ in.}^3} - \frac{(624.21\text{k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$= 3.29\text{ ksi}$ $< 4.225\text{ ksi}$ OK

@ Debond Location 2 + Transfer (312 in. from Beam End)

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

$w_b = 1.021\text{ k / ft.}$

$L_c = 12\text{ ft.}$

$L_b = 159.25\text{ ft.}$

x = distance from (debond location 2 + transfer) to lifting loop location

= (312 in. – 144 in.)(1 ft. / 12 in.)

= 14 ft.

$$M_{bts} = -\frac{(1.021\,k/ft.)(12\,ft.)^2}{2} + \frac{(1.021\,k/ft.)(14\,ft.)}{2}(159.25\,ft. - 2(12\,ft.) - 11\,ft.)$$

= 814.5 k-ft.

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

A_{ps} = 10.71 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pES} = 15.03 ksi

$$F_t = 10.71\,in.^2(202.3\,ksi - 15.03\,ksi)$$

= 2006 kips

e = 19.90 in. (See App. B)

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

$$= \frac{2006\,k}{980\,in.^2} - \frac{(2006\,k)(19.90\,in.)}{14648.6\,in.^3} + \frac{(814.5\,k-ft.)\left(\frac{12\,in.}{ft.}\right)}{14648.6\,in.^3}$$

= -0.01 ksi > -0.612 ksi OK

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

$$= \frac{2006\,k}{980\,in.^2} + \frac{(2006\,k)(19.90\,in.)}{21237.8\,in.^3} - \frac{(814.5\,k-ft.)\left(\frac{12\,in.}{ft.}\right)}{21237.8\,in.^3}$$

= 3.47 ksi < 4.225 ksi OK

@ Harping Point, 0.4L (764.4 in. from Beam End)

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

w_b = 1.021 k / ft.

L_c = 12 ft.

L_b = 159.25 ft.

x = distance from debond location 1 + transfer to lifting loop location

= (764.4 in. – 144 in.)(1 ft. / 12 in.)

= 51.7 ft.

$$M_{bts} = -\frac{(1.021\text{ k / ft.})(12\text{ ft.})^2}{2} + \frac{(1.021\text{ k / ft.})(51.7\text{ ft.})}{2}(159.25\text{ ft.} - 2(12\text{ ft.}) - 51.7\text{ ft.})$$

= 2131.6 k-ft.

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

A_{ps} = 11.28 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pES} = 15.03 ksi

$$F_t = 11.28\text{ in.}^2(202.3\text{ ksi} - 15.03\text{ ksi})$$

= 2112 kips

e = 22.81 in. (See App. B)

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

$$= \frac{2112\text{ k}}{980\text{ in.}^2} - \frac{(2112\text{ k})(22.81\text{ in.})}{14648.6\text{ in.}^3} + \frac{(2131.6\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

= 0.613 ksi > -0.612 ksi OK

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

$$= \frac{2112\text{ k}}{980\text{ in.}^2} + \frac{(2112\text{ k})(22.81\text{ in.})}{21237.8\text{ in.}^3} - \frac{(2131.6\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

= 3.22 ksi < 4.225 ksi OK

Service State Limit Stresses (5.9.2.3.2)

Service stresses after losses (5.9.2.3.2)

Compression (For Service I load combination):

$0.60\phi_w f'_c = 0.60(1.0)(8.5\text{ ksi}) = 5.10\text{ ksi}$ (a)

$0.45f'_c = 0.45(8.5\text{ ksi}) = 3.83\text{ ksi}$ (b)

Tension (For Service III load combination):

$0.19\sqrt{f'_c} = 0.19\sqrt{8.5\text{ ksi}} = -0.55\text{ ksi}$

@ Debond Location 1 (240 in. from beam end)

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

$A_{ps} = 9.55\text{ in.}^2$ (See App. B)

$f_{pbt} = 202.3\text{ ksi}$

$\Delta f_{pT} = 40.97\text{ ksi}$

$F_s = (9.55\text{ in.}^2)(202.3\text{ ksi} - 40.97\text{ ksi})$

$= 1540\text{ kips}$

$e = 18.61\text{ in.}$ (See App. B)

$M_{DC1} = 2397.6\text{ k-ft.}$

$M_{DC2} = 184.0\text{ k-ft.}$

$M_{DW} = 339.0\text{ k-ft.}$

$M_{LL+IM} = 1273.2\text{ k-ft.}$

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{(M_{DC2} + M_{DW} + M_{LL+IM})}{S'_t}$$ (a)

$$= \frac{1540\text{ k}}{980\text{ in.}^2} - \frac{(1540\text{ k})(18.61\text{ in.})}{14648.6\text{ in.}^3} + \frac{2397.6\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{(184.0\text{ k}-\text{ft.} + 339.0\text{ k}-\text{ft.} + 1273.2\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

$= 2.01$ ksi ≤ 5.10 ksi OK

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{(M_{DC2} + M_{DW})}{S'_t} \quad \text{(b)}$$

$$= \frac{1540\text{ k}}{980\text{ in.}^2} - \frac{(1540\text{ k})(18.61\text{ in.})}{14648.6\text{ in.}^3} + \frac{2397.6\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{(184\text{ k}-\text{ft.} + 339\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

$= 1.70$ ksi ≤ 3.83 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{M_{DC2} + M_{DW}}{S'_b} - 0.8\frac{M_{LL+IM}}{S'_b}$$

$$= \frac{1540\text{ k}}{980\text{ in.}^2} + \frac{(1540\text{ k})(18.61\text{ in.})}{21237.8\text{ in.}^3} - \frac{2397.6\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$- \frac{(184.0\text{ k}-\text{ft.} + 339.0\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.7\text{ in.}^3} - 0.8\frac{1273.2\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.7\text{ in.}^3}$$

$= 0.95$ ksi > -0.55 ksi OK

@ Debond Location 1 + Transfer / Debond Location 2 (276 in. from Beam End)

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

$A_{ps} = 10.1$ in.^{2} (See App. B)

$f_{pbt} = 202.3$ ksi

$\Delta f_{pT} = 40.97$ ksi

$$F_s = \left(10.1\text{ in.}^2\right)\left(202.3\text{ ksi} - 40.97\text{ ksi}\right)$$

$$= 1628\text{ kips}$$

e = 19.31 in. (See App. B)

M_{DC1} = 2709.4 k-ft.

M_{DC2} = 206.1 k-ft.

M_{DW} = 379.7 k-ft.

M_{LL+IM} = 1429.0 k-ft.

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW} + M_{LL+IM}\right)}{S'_t} \quad \text{(a)}$$

$$= \frac{1628\text{ k}}{980\text{ in.}^2} - \frac{\left(1628\text{ k}\right)\left(19.31\text{ in.}\right)}{14648.6\text{ in.}^3} + \frac{2709.4\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{\left(206.1\text{ k} - \text{ft.} + 379.7\text{ k} - \text{ft.} + 1429.0\text{ k} - \text{ft.}\right)\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 2.22 ksi ≤ 5.10 ksi OK

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW}\right)}{S'_t} \quad \text{(b)}$$

$$= \frac{1628\text{ k}}{980\text{ in.}^2} - \frac{\left(1628\text{ k}\right)\left(19.31\text{ in.}\right)}{14648.6\text{ in.}^3} + \frac{2709.4\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{\left(206.1\text{ k} - \text{ft.} + 379.7\text{ k} - \text{ft.}\right)\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 1.88 ksi ≤ 3.83 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{M_{DC2} + M_{DW}}{S'_b} - 0.8\frac{M_{LL+IM}}{S'_b}$$

$$= \frac{1628\text{ k}}{980\text{ in.}^2} + \frac{(1628\text{ k})(19.31\text{ in.})}{21237.8\text{ in.}^3} - \frac{2709.4\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$-\frac{(206.1\text{ k}-\text{ft.} + 379.7\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3} - 0.8\frac{1429.0\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

= 0.91 ksi > -0.55 ksi OK

@ Debond Location 2 + Transfer (312 in. from Beam End)

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

A_{ps} = 10.71 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

F_s = $(10.71\text{ in.}^2)(202.3\text{ ksi} - 40.97\text{ ksi})$

= 1729 kips

e = 19.9 in. (See App. B)

M_{DC1} = 3005.3 k-ft.

M_{DC2} = 226.5 k-ft.

M_{DW} = 417.3 k-ft.

M_{LL+IM} = 1573.7 k-ft.

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

$$= \frac{1729\text{ k}}{980\text{ in.}^2} - \frac{(1729\text{ k})(19.9\text{ in.})}{14648.6\text{ in.}^3} + \frac{3005.3\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+\frac{(226.9\text{ k}-\text{ft.} + 417.3\text{ k}-\text{ft.} + 1573.7\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 2.41 ksi ≤ 5.10 ksi OK

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{(M_{DC2} + M_{DW})}{S'_t} \qquad \text{(b)}$$

$$= \frac{1729\text{ k}}{980\text{ in.}^2} - \frac{(1729\text{ k})(19.9\text{ in.})}{14648.6\text{ in.}^3} + \frac{3005.3\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{(226.9\text{ k}-\text{ft.} + 417.3\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

$= 2.03$ ksi $\leq$ 3.83 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{M_{DC2} + M_{DW}}{S'_b} - 0.8\frac{M_{LL+IM}}{S'_b}$$

$$= \frac{1729\text{ k}}{980\text{ in.}^2} + \frac{(1729\text{ k})(19.31\text{ in.})}{21237.8\text{ in.}^3} - \frac{3005.3\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$- \frac{(226.9\text{ k}-\text{ft.} + 417.3\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3} - 0.8\frac{1573.7\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

$= 0.92$ ksi $>$ -0.55 ksi OK

@ Harping Point, 0.4L (764.4 in. from Beam End)

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

$A_{ps} =$ 11.28 in.^{2} (See App. B)

$f_{pbt} =$ 202.3 ksi

$\Delta f_{pT} =$ 40.97 ksi

$F_s = (11.28\text{ in.}^2)(202.3\text{ ksi} - 40.97\text{ ksi})$

= 1820 kips

e = 22.81 in. (See App. B)

M_{DC1} = 5381.2 k-ft.

M_{DC2} = 336.8 k-ft.

M_{DW} = 620.4 k-ft.

M_{LL+IM} = 2526.3 k-ft.

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW} + M_{LL+IM}\right)}{S'_t} \qquad \text{(a)}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} - \frac{(1820\text{ k})(22.81\text{ in.})}{14648.6\text{ in.}^3} + \frac{5381.2\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{(336.8\text{ k} - \text{ft.} + 620.4\text{ k} - \text{ft.} + 2526.3\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 4.27 ksi ≤ 5.10 ksi OK

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW}\right)}{S'_t} \qquad \text{(b)}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} - \frac{(1820\text{ k})(22.81\text{ in.})}{14648.6\text{ in.}^3} + \frac{5381.2\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{(336.8\text{ k} - \text{ft.} + 620.4\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 3.66 ksi ≤ 3.83 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{M_{DC2} + M_{DW}}{S'_b} - 0.8\frac{M_{LL+IM}}{S'_b}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} + \frac{(1820\text{ k})(22.81\text{ in.})}{21237.8\text{ in.}^3} - \frac{5381.2\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$- \frac{(336.8\text{ k} - \text{ft.} + 620.4\text{ k} - \text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3} - 0.8\frac{2526.3\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

= -0.43 ksi > -0.55 ksi OK

@ Center of Span, 0.5L (955.5 in. from Beam End)

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

A_{ps} = 11.28 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

$$F_s = \left(11.28\text{ in.}^2\right)\left(202.3\text{ ksi} - 40.97\text{ ksi}\right)$$

= 1820 kips

e = 22.81 in. (See App. B)

M_{DC1} = 5605.4 k-ft.

M_{DC2} = 300.7 k-ft.

M_{DW} = 553.9 k-ft.

M_{LL+IM} = 2480.5 k-ft.

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW} + M_{LL+IM}\right)}{S'_t} \quad \text{(a)}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} - \frac{\left(1820\text{ k}\right)\left(22.81\text{ in.}\right)}{14648.6\text{ in.}^3} + \frac{5605.4\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+ \frac{\left(300.7\text{ k} - \text{ft.} + 553.9\text{ k} - \text{ft.} + 2480.5\text{ k} - \text{ft.}\right)\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 4.42 ksi ≤ 5.10 ksi OK

$$f_t = \frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW}\right)}{S'_t} \quad \text{(b)}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} - \frac{\left(1820\text{ k}\right)\left(22.81\text{ in.}\right)}{14648.6\text{ in.}^3} + \frac{5605.4\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}$$

$$+\frac{(300.7\text{ k}-\text{ft.}+553.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 3.82 ksi ≤ 3.83 ksi OK

$$f_b = \frac{F_S}{A}+\frac{F_S e}{S_b}-\frac{M_{DC1}}{S_b}-\frac{M_{DC2}+M_{DW}}{S'_b}-0.8\frac{M_{LL+IM}}{S'_b}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2}+\frac{(1820\text{ k})(22.81\text{ in.})}{21237.8\text{ in.}^3}-\frac{5605.4\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$-\frac{(300.7\text{ k}-\text{ft.}+553.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}-0.8\frac{2480.5\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

= -0.50 ksi > -0.55 ksi OK

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

Fatigue stresses after losses (5.5.3.1)

Compression (For Fatigue I load combination):

$0.40f'_c$ = 0.40(8.5 ksi) = 3.4 ksi

Tension limit for determination of uncracked section:

$0.095\sqrt{f'_c} = 0.095\sqrt{8.5\text{ ksi}}$ = -0.277 ksi

@ Debond Location 1 (240 in. from beam end)

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

A_{ps} = 9.55 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

F_s = $(9.55\text{ in.}^2)(202.3\text{ ksi}-40.97\text{ ksi})$

= 1540 kips

e = 18.61 in. (See App. B)

M_{DC1} = 2397.6 k-ft.

M_{DC2} = 184.0 k-ft.

M_{DW} = 339.0 k-ft.

M_{FL+IM} = 378.2 k-ft.

$$f_t = 0.5\left[\frac{F_S}{A}-\frac{F_Se}{S_t}+\frac{M_{DC1}}{S_t}+\frac{(M_{DC2}+M_{DW})}{S'_t}\right]+1.75\frac{M_{FL+IM}}{S'_t} \quad \text{(a)}$$

$$= 0.5\left[\frac{1540\text{ k}}{980\text{ in.}^2}-\frac{(1540\text{ k})(18.61\text{ in.})}{14648.6\text{ in.}^3}+\frac{2397.6\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}\right]$$

$$+0.5\frac{(184.0\text{ k}-\text{ft.}+339.0\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}+1.75\frac{378.2\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 1.01 ksi ≤ 3.4 ksi OK

$$f_b = \frac{F_S}{A}+\frac{F_Se}{S_b}-\frac{M_{DC1}}{S_b}-\frac{(M_{DC2}+M_{DW})}{S'_b}-1.75\frac{M_{FL+IM}}{S'_b}$$

$$= \frac{1540\text{ k}}{980\text{ in.}^2}+\frac{(1540\text{ k})(18.61\text{ in.})}{21237.8\text{ in.}^3}-\frac{2397.6\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$-\frac{(184.0\text{ k}-\text{ft.}+339.0\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}-1.75\frac{378.2\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

= 1.09 ksi > -0.277 ksi OK

@ Debond Location 1 + Transfer / Debond Location 2 (276 in. from Beam End)

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

A_{ps} = 10.1 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

F_s = $\left(10.1\,\text{in.}^2\right)\left(202.3\,\text{ksi} - 40.97\,\text{ksi}\right)$

= 1628 kips

e = 19.31 in. (See App. B)

M_{DC1} = 2709.4 k-ft.

M_{DC2} = 206.1 k-ft.

M_{DW} = 379.7 k-ft.

M_{FL+IM} = 422.8 k-ft.

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW}\right)}{S'_t}\right] + 1.75\frac{M_{FL+IM}}{S'_t} \quad \text{(a)}$$

$$= 0.5\left[\frac{1628\,\text{k}}{980\,\text{in.}^2} - \frac{\left(1628\,\text{k}\right)\left(19.31\,\text{in.}\right)}{14648.6\,\text{in.}^3} + \frac{2709.4\,\text{k} - \text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{14648.6\,\text{in.}^3}\right]$$

$$+0.5\frac{\left(206.1\,\text{k} - \text{ft.} + 379.7\,\text{k} - \text{ft.}\right)\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{49907.41\,\text{in.}^3} + 1.75\frac{422.8\,\text{k} - \text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{49907.41\,\text{in.}^3}$$

= 1.12 ksi ≤ 3.4 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{\left(M_{DC2} + M_{DW}\right)}{S'_b} - 1.75\frac{M_{FL+IM}}{S'_b}$$

$$= \frac{1628\,\text{k}}{980\,\text{in.}^2} + \frac{\left(1628\,\text{k}\right)\left(19.31\,\text{in.}\right)}{21237.8\,\text{in.}^3} - \frac{2709.4\,\text{k} - \text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{21237.8\,\text{in.}^3}$$

$$-\frac{\left(206.1\,\text{k} - \text{ft.} + 379.7\,\text{k} - \text{ft.}\right)\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{29802.74\,\text{in.}^3} - 1.75\frac{422.8\,\text{k} - \text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{29802.74\,\text{in.}^3}$$

= 1.08 ksi > -0.277 ksi OK

@ Debond Location 2 + Transfer (312 in. from Beam End)

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

A_{ps} = 10.71 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

$$F_s = \left(10.71\,\text{in.}^2\right)\left(202.3\,\text{ksi} - 40.97\,\text{ksi}\right)$$

$$= 1729\ \text{kips}$$

e = 19.9 in. (See App. B)

M_{DC1} = 3005.3 k-ft.

M_{DC2} = 226.9 k-ft.

M_{DW} = 417.3 k-ft.

M_{FL+IM} = 463.8 k-ft.

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{\left(M_{DC2} + M_{DW}\right)}{S'_t}\right] + 1.75\frac{M_{FL+IM}}{S'_t} \qquad \text{(a)}$$

$$= 0.5\left[\frac{1729\,\text{k}}{980\,\text{in.}^2} - \frac{\left(1729\,\text{k}\right)\left(19.9\,\text{in.}\right)}{14648.6\,\text{in.}^3} + \frac{3005.3\,\text{k}-\text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{14648.6\,\text{in.}^3}\right]$$

$$+0.5\frac{\left(226.9\,\text{k}-\text{ft.} + 417.3\,\text{k}-\text{ft.}\right)\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{49907.41\,\text{in.}^3} + 1.75\frac{463.8\,\text{k}-\text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{49907.41\,\text{in.}^3}$$

= 1.21 ksi ≤ 3.4 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{\left(M_{DC2} + M_{DW}\right)}{S'_b} - 1.75\frac{M_{FL+IM}}{S'_b}$$

$$= \frac{1729\,\text{k}}{980\,\text{in.}^2} + \frac{\left(1729\,\text{k}\right)\left(19.9\,\text{in.}\right)}{21237.8\,\text{in.}^3} - \frac{3005.3\,\text{k}-\text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{21237.8\,\text{in.}^3}$$

$$-\frac{\left(226.9\,\text{k}-\text{ft.} + 417.3\,\text{k}-\text{ft.}\right)\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{29802.74\,\text{in.}^3} - 1.75\frac{463.8\,\text{k}-\text{ft.}\left(\frac{12\,\text{in.}}{\text{ft.}}\right)}{29802.74\,\text{in.}^3}$$

= 1.10 ksi > -0.277 ksi OK

@ Harping Point, 0.4L (764.4 in. from Beam End)

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

$A_{ps} = 11.28 \text{ in.}^2$ (See App. B)

$f_{pbt} = 202.3 \text{ ksi}$

$\Delta f_{pT} = 40.97 \text{ ksi}$

$$F_s = \left(11.28 \text{ in.}^2\right)\left(202.3 \text{ ksi} - 40.97 \text{ ksi}\right)$$

$= 1820 \text{ kips}$

$e = 22.81 \text{ in.}$ (See App. B)

$M_{DC1} = 5381.2 \text{ k-ft.}$

$M_{DC2} = 336.8 \text{ k-ft.}$

$M_{DW} = 620.4 \text{ k-ft.}$

$M_{FL+IM} = 722.2 \text{ k-ft.}$

$$f_t = 0.5\left[\frac{F_S}{A} - \frac{F_S e}{S_t} + \frac{M_{DC1}}{S_t} + \frac{(M_{DC2} + M_{DW})}{S'_t}\right] + 1.75\frac{M_{FL+IM}}{S'_t} \quad \text{(a)}$$

$$= 0.5\left[\frac{1820 \text{ k}}{980 \text{ in.}^2} - \frac{(1820 \text{ k})(22.81 \text{ in.})}{14648.6 \text{ in.}^3} + \frac{5381.2 \text{ k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{14648.6 \text{ in.}^3}\right]$$

$$+0.5\frac{(336.8 \text{ k} - \text{ft.} + 620.4 \text{ k} - \text{ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{49907.41 \text{ in.}^3} + 1.75\frac{722.2 \text{ k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{49907.41 \text{ in.}^3}$$

$= 1.83 \text{ ksi} \leq 3.4 \text{ ksi}$ OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{(M_{DC2} + M_{DW})}{S'_b} - 1.75\frac{M_{FL+IM}}{S'_b}$$

$$= \frac{1820 \text{ k}}{980 \text{ in.}^2} + \frac{(1820 \text{ k})(22.81 \text{ in.})}{21237.8 \text{ in.}^3} - \frac{5381.2 \text{ k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{21237.8 \text{ in.}^3}$$

$$-\frac{(336.8\text{ k}-\text{ft.}+620.4\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}-1.75\frac{722.2\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

= -0.12 ksi < -0.277 ksi OK

@ Center of Span, 0.5L (955.5 in. from Beam End)

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

A_{ps} = 11.28 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT} = 40.97 ksi

F_s = $\left(11.28\text{ in.}^2\right)(202.3\text{ ksi}-40.97\text{ ksi})$

= 1820 kips

e = 22.81 in. (See App. B)

M_{DC1} = 5605.4 k-ft.

M_{DC2} = 300.7 k-ft.

M_{DW} = 553.9 k-ft.

M_{FL+IM} = 704.0 k-ft.

$$f_t = 0.5\left[\frac{F_S}{A}-\frac{F_S e}{S_t}+\frac{M_{DC1}}{S_t}+\frac{(M_{DC2}+M_{DW})}{S'_t}\right]+1.75\frac{M_{FL+IM}}{S'_t} \quad \text{(a)}$$

$$= 0.5\left[\frac{1820\text{ k}}{980\text{ in.}^2}-\frac{(1820\text{ k})(22.81\text{ in.})}{14648.6\text{ in.}^3}+\frac{5605.4\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{14648.6\text{ in.}^3}\right]$$

$$+0.5\frac{(300.7\text{ k}-\text{ft.}+553.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}+1.75\frac{704.0\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{49907.41\text{ in.}^3}$$

= 2.21 ksi ≤ 3.4 ksi OK

$$f_b = \frac{F_S}{A} + \frac{F_S e}{S_b} - \frac{M_{DC1}}{S_b} - \frac{(M_{DC2} + M_{DW})}{S'_b} - 1.75\frac{M_{FL+IM}}{S'_b}$$

$$= \frac{1820\text{ k}}{980\text{ in.}^2} + \frac{(1820\text{ k})(22.81\text{ in.})}{21237.8\text{ in.}^3} - \frac{5605.4\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{21237.8\text{ in.}^3}$$

$$- \frac{(300.7\text{ k}-\text{ft.} + 553.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3} - 1.75\frac{705.4\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{29802.74\text{ in.}^3}$$

$= -0.20$ ksi $>$ -0.277 ksi OK

Strength I Moment

The maximum factored Strength I moment occurs at midspan. This is because the dead load of the beam and slab, which are simply supported, contribute a larger portion of the total moment than the continuous loads.

M_u = $1.25(M_{DC1} + M_{DC2}) + 1.5M_{DW} + 1.75(M_{LL+IM})$
= 1.25(5605.4 k-ft. + 300.7 k-ft.) + 1.5(553.9 k-ft.) + 1.75(2480.5 k-ft.)
= 12554 k-ft.

Factored Flexural Resistance

Determine location of neutral axis

$a = \beta_1 c$

Check neutral axis in slab

c, neutral axis in slab

$$= \frac{A_{ps}f_{pu}}{\alpha_1 f'_{c,slab}\beta_1 b_{slab} + kA_{ps}\frac{f_{pu}}{d_p}} \quad \text{(Eq. 5.6.3.1.1-4)}$$

A_{ps} = 11.284 in.^{2}

f_{pu} = 270 ksi

α_1 = 0.85 (5.6.2.2)

β_1 = 0.85

$f'_{c,slab}$ = 4 ksi

b_{slab} = 84 in.

k = 0.28 for low-relaxation strands (Table C5.7.3.1.1-1)

d_p = 72 in. beam + 8 in. slab – 6.58 ht. to centroid of strand group

= 73.42 in.

$$c = \frac{\left(11.284\text{ in.}^2\right)\left(270\text{ ksi}\right)}{0.85\left(4\text{ ksi}\right)\left(0.85\right)\left(84\text{ in.}\right)+0.28\left(11.284\text{ in.}^2\right)\frac{270\text{ ksi}}{73.42\text{ in.}}}$$

= 11.98 in.

a = 0.85(11.98 in.)

= 10.18 in. > t_{slab} = 8 in., therefore the neutral axis is not in the deck. The top flange of the beam is 6.06 in. thick, making the (slab + top flange) thickness equal to 14.06 in., which is greater than 10.18 in. Therefore, the neutral axis is likely in the top flange.

Check neutral axis in top flange

c, neutral axis in top flange of beam

$$= \frac{A_{ps}f_{pu}-\alpha_1 f'_{c,slab}\left(b_{slab}-b_{tf}\right)t_{slab}}{\alpha_1 f'_{c,slab}\,\beta_1 b_{tf}+kA_{ps}\frac{f_{pu}}{d_p}} \qquad \text{(Eq. 5.6.3.1.1-3)}$$

A_{ps} = 11.284 in.^{2}

f_{pu} = 270 ksi

α_1 = 0.85 (5.6.2.2)

$f'_{c,slab}$ = 4 ksi

b_{slab} = 84 in.

b_{tf} = $\frac{5362\text{ ksi}}{3987\text{ ksi}}\left(24\text{ in.}\right)$

= 32.43 in., note that this is the top flange width transformed to be consistent with the slab properties

t_{slab} = 8 in.

k = 0.28 for low-relaxation strands (Table C5.7.3.1.1-1)

d_p = 72 in. beam + 8 in. slab – 6.58 height to centroid of strand group

$= 73.42 \text{ in.}$

$$= \frac{(11.284 \text{ in.}^2)(270 \text{ ksi}) - 0.85(4 \text{ ksi})(84 \text{ in.} - 32.43 \text{ in.})(8 \text{ in.})}{0.85(4 \text{ ksi})(0.85)(32.43 \text{ in.}) + 0.28(11.284 \text{ in.}^2)\frac{270 \text{ ksi}}{73.42 \text{ in.}}}$$

$= 15.61 \text{ in.}$

$a = 0.85(15.61 \text{ in.})$

$= 13.27 \text{ in.} < t_{slab} + t_{tf} = 8 \text{ in.} + 6.06 \text{ in.} = 14.06 \text{ in.}$, neutral axis in top flange

Calculate Flexural Resistance

$M_r = \phi M_n$ (Eq. 5.6.3.2.1-1)

Where:

M_n, neutral axis in top flange of beam

$$= A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 f'_c\left(b_{slab} - b_{tf}\right)t_{slab}\left(\frac{a}{2} - \frac{t_{slab}}{2}\right) \quad \text{(Eq. 5.6.3.2.2-1)}$$

In which:

$A_{ps} = 11.284 \text{ in.}^2$

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(Eq. 5.6.3.1.1-1)}$$

$f_{pu} = 270 \text{ ksi}$

$k = 0.28$

$c = 15.61 \text{ in.}$

$d_p = 73.42 \text{ in.}$

$$f_{ps} = (270 \text{ ksi})\left(1 - 0.28\frac{15.61 \text{ in.}}{73.42 \text{ in.}}\right)$$

$= 253.92 \text{ ksi}$

$$\phi = 0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0 \quad \text{(Eq. 5.5.4.2-1)}$$

$$\varepsilon_t = \frac{0.003\left(h_{beam} + t_{fillet} + t_{slab} - 2\text{ in. clear} - 0.5(d_{strand}) - c\right)}{c}$$

$$= \frac{0.003\left(72\text{ in.} + 0\text{ in.} + 8\text{ in.} - 2\text{ in. clear} - 0.5(0.6\text{ in.}) - 15.61\text{ in.}\right)}{15.61\text{ in.}}$$

$$= 0.012$$

ε_{cl} = 0.002 (5.6.2.1)

ε_{tl} = 0.005 (5.6.2.1)

$$\phi = 0.75 + \frac{0.25\left(0.012 - 0.002\right)}{\left(0.005 - 0.002\right)}$$ (Eq. 5.5.4.2-1)

= 1.58 > 1.0, therefore ϕ = 1.0

$$\phi M_n = 1.0[\left(11.284\text{ in.}^2\right)\left(253.92\text{ ksi}\right)\left(73.42\text{ in.} - \frac{13.27\text{ in.}}{2}\right)$$

$$+ 0.85\left(4\text{ ksi}\right)\left(84\text{ in.} - 32.43\text{ in.}\right)\left(8\text{ in.}\right)\left(\frac{13.27\text{ in.}}{2} - \frac{8\text{ in.}}{2}\right)]$$

= 195060 k – in.

= 16255 k -ft. > 12554 k-ft. OK

<u>Minimum Reinforcement</u> <u>(5.6.3.3)</u>

$M_r \geq M_{cr}$ (5.6.3.3)

In which:

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{DC1}\left(\frac{S'_b}{S_b} - 1\right)\right]$$ (Eq. 5.6.3.3-1)

Where:

$f_r = 0.24\sqrt{8.5\text{ ksi}}$ (5.4.2.6)

= 0.70 ksi

S_b = 21237.8 in.^{3}

S'_b = 29802.74 in.^{3}

M_{DC1} = 5605.4 kip-ft.

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

F_S = 1820 k

A = 980 in.^{2}

e = 22.81 in.

γ_1 = 1.6

γ_2 = 1.1

γ_3 = 1.00

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

$$= \frac{1820\text{ k}}{980\text{ in.}^2} + \frac{(1820\text{ k})(22.81\text{ in.})}{21237.8\text{ in.}^3}$$

$$= 3.81\text{ ksi}$$

M_{cr} =

$$1.00\left[\left(1.6(0.7\text{ ksi}) + 1.1(3.81\text{ ksi})\right)\left(29802.74\text{ in.}^3\right) - \left(5605.4\text{ k} - \text{ft.}\right)\left(\frac{29802.74\text{ in.}^3}{21237.8\text{ in.}^3} - 1\right)\right]$$

= 10930 k-ft. < ϕM_n = 16255 k-ft. OK

Negative Moment Region Design *(5.7.3)*

<u>Calculate Strength I Moment</u>

At the centerline of pier, the factored Strength I moment is:

M_u = 1.25(M_{DC1} + M_{DC2}) + 1.5M_{DW} + 1.75(M_{LL+IM})

= 1.25(0 k-ft. + -601.4 k-ft.) + 1.5(-1107.8 k-ft.) + 1.75(-2588.6 k-ft.)

= -6944 k-ft.

= 83328 k-in.

Estimate Negative Moment Reinforcement

$$R_n = \frac{M_u}{\phi b d_s^{\,2}}$$

In which:

M_u = 83328 k-in.

b = 38 in.

$$d_s = h_{beam} + t_{fillet} + \frac{t_{slab}}{2}$$

$$= 72\text{ in.} + 0\text{ in.} + \frac{8\text{ in.}}{2}$$

= 76.0 in.

ϕ = 0.9

$$R_n = \frac{83328\text{ k} - \text{in.}}{(0.9)(38\text{ in.})(76.0\text{ in.})^2}$$

= 0.422 ksi

$$\rho = \frac{\alpha_1 f'_{c,beam}}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{\alpha_1 f'_{c,beam}}}\right]$$

In which:

α_1 = 0.85

$f'_{c,beam}$ = 8.5 ksi

f_y = 60 ksi

R_n = 0.422 ksi

$$\rho = \frac{0.85(8.5\text{ ksi})}{60\text{ ksi}}\left[1 - \sqrt{1 - \frac{2(0.422\text{ ksi})}{(0.85)(8.5\text{ ksi})}}\right]$$

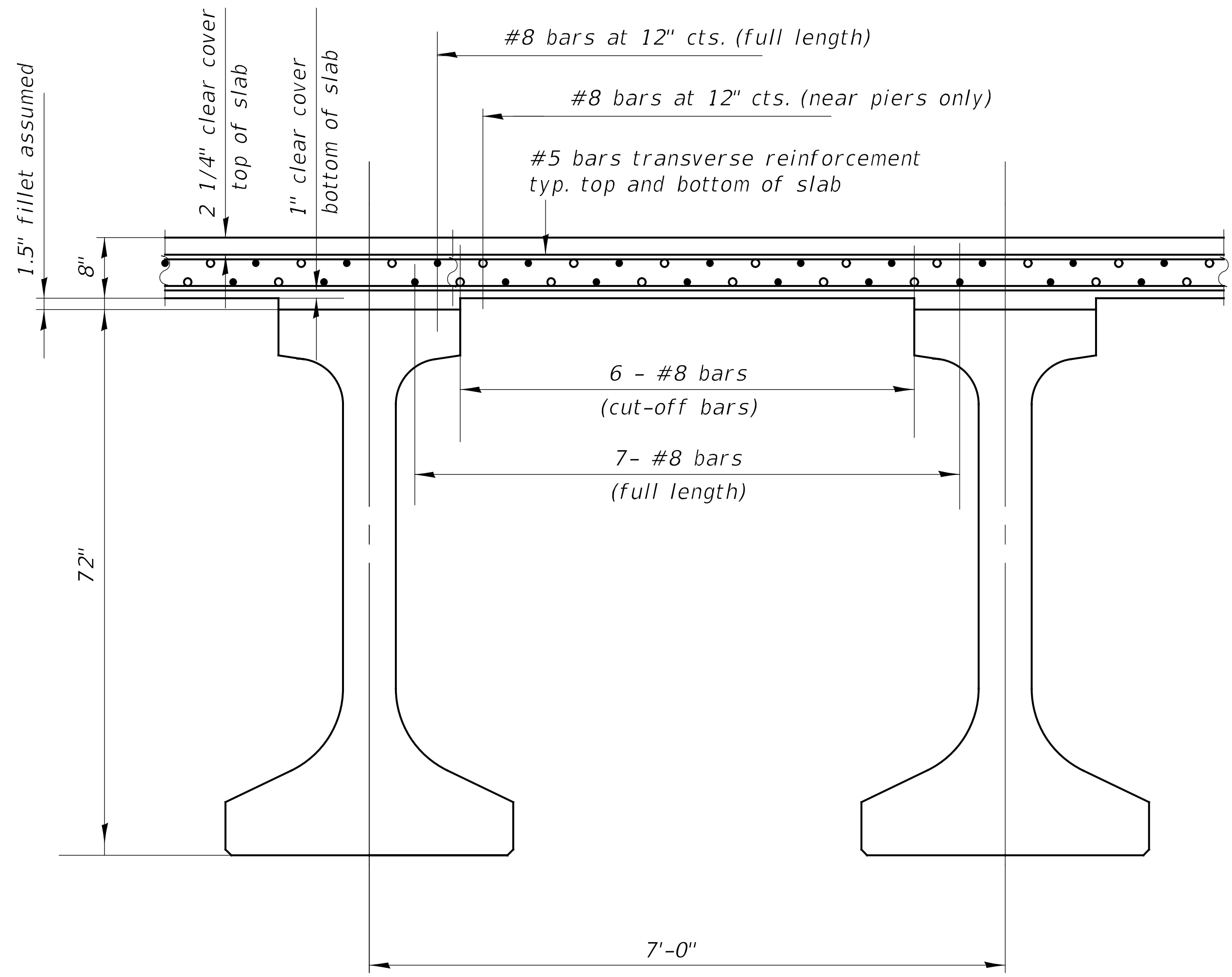
= 0.00725

$A_s = \rho b d_s$

= 0.00725(38 in.)(76 in.)

= 20.94 in.^{2}

Determine Trial Reinforcement Configuration



CROSS SECTION AT PIER

Area of Reinforcement:

Top (full length)	=	#8 @ 12”	=	(0.79 in.^{2} / ft.)(7 ft. EFW)	=	5.53 in.^{2}
Top (cut-off)	=	#8 @ 12”	=	(0.79 in.^{2} / ft.)(7 ft. EFW)	=	5.53 in.^{2}
Bottom (full length)	=	7 - #8	=	7(0.79 in.^{2})	=	5.53 in.^{2}
Bottom (cut-off)	=	6 - #8	=	6(0.79 in.^{2})	=	4.74 in.^{2}

Totals:

A_s (full length) = 5.53 in.^{2} + 5.53 in.^{2}

= 11.06 in.^{2}

A_s (total) = 5.53 in.^{2} + 5.53 in.^{2} + 5.53 in.^{2} + 4.74 in.^{2}

= 21.33 in.^{2} ≥ 20.94 in.^{2} OK for estimate

Check minimum 1/3 reinforcement to continue past point of inflection (5.10.8.1.2c)

$$\frac{A_s(\text{full length})}{A_s(\text{total})} = \frac{11.03\text{ in.}^2}{21.33\text{ in.}^2}$$

$$= 0.52 \geq 0.333$$ OK

Because less than half of the bars are being cut off, the requirement of 5.10.8.1.2 that no more than half the bars be cut off in the same location is met, and there is no need to stagger the reinforcement cutoffs.

Transverse slab reinforcement consists of #5 bars, with spacing as determined by the deck design chart in ABD 15.2.

Center of Gravity of Reinforcement:

	A_s	d_s	A_sd_s
Top (full length)	5.53 in.^{2}	76.63 in.	423.74 in.^{3}
Top (cut-off)	5.53 in.^{2}	76.63 in.	423.74 in.^{3}
Bottom (full length)	5.53 in.^{2}	74.13 in.	409.91 in.^{3}
Bottom (cut-off)	4.74 in.^{2}	74.13 in.	351.35 in.^{3}
	21.33 in.^{2}		1608.74 in.^{3}

$$d_s = \frac{1608.74\text{ in.}^3}{21.33\text{ in.}^2} = 75.42\text{ in.}$$

<u>Factored Flexural Resistance</u> (5.6.3.2.1)

Determine location of neutral axis

$a = \beta_1 c$

Check neutral axis in bottom flange

c, neutral axis in bottom flange

$$= \frac{A_s f_y}{\alpha_1 f'_{c,beam} \beta_1 b_{bf}} \qquad \text{(Eq. 5.6.3.1.1-4)}$$

A_s = 21.33 in.^{2}

f_y = 60 ksi

α_1 = 0.85 (5.6.2.2)

β_1 = $0.85 – 0.05(f'_c – 4.0) > 0.65$ (5.6.2.2)

= $0.85 – 0.05(8.5 – 4.0)$

= $0.625 < 0.65$, therefore use 0.65

$f'_{c,beam}$ = 8.5 ksi

b_{bf} = 38 in.

$$c = \frac{(21.33\text{ in.}^2)(60\text{ ksi})}{0.85(8.5\text{ ksi})(0.65)(38\text{ in.})}$$

= 7.17 in.

a = 0.65(7.17 in.)

= 4.66 in. < t_{bf} = 9.5 in., therefore the neutral axis is in the bottom flange

Calculate Flexural Resistance

$M_r = \phi M_n$ (Eq. 5.6.3.2.1-1)

Where:

M_n, neutral axis in bottom flange of beam

$$= A_s f_y \left(d_s - \frac{a}{2}\right) \qquad \text{(Eq. 5.6.3.2.2-1)}$$

In which:

A_s = 21.33 in.^{2}

f_y = 60 ksi

d_s = 75.42 in.

a = 4.66 in.

$$M_n = \left(21.33\text{ in.}^2\right)\left(60\text{ ksi}\right)\left(75.42\text{ in.} - \frac{4.66\text{ in.}}{2}\right)\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)$$

$$= 7794\text{ k-ft.}$$

$$\phi = 0.75 \le \phi = 0.75 + \frac{0.15\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \le 0.9 \qquad \text{(Eq. 5.5.4.2-1)}$$

$$\varepsilon_t = \frac{0.003\left(h_{beam} + t_{fillet} + t_{slab} - 2.25\text{ in. clear} - 0.625\text{ in.} - 0.5(d_{top\ slab\ reinf.}) - c\right)}{c}$$

$$= \frac{0.003\left(72\text{ in.} + 0\text{ in.} + 8\text{ in.} - 2.25\text{ in. clear} - 0.625\text{ in.} - 0.5(1.0\text{ in.}) - 7.17\text{ in.}\right)}{7.17\text{ in.}}$$

$$= 0.029$$

ε_{cl} = 0.002 (5.6.2.1)

ε_{tl} = 0.005 (5.6.2.1)

$$\phi = 0.75 + \frac{0.15\left(0.029 - 0.002\right)}{\left(0.005 - 0.002\right)} \qquad \text{(Eq. 5.5.4.2-1)}$$

= 2.1 > 0.9, therefore ϕ = 0.9

ϕM_n = 0.9(7794 k-ft.)

= 7015 k -ft. > 6944 k-ft. OK

Note that the applied moments seem very close to the moment capacity of the section. The applied moments were conservatively taken at the centerline of the pier. The actual design section may be taken at the face of the pier, where there is considerably less moment.

<u>Minimum Reinforcement</u> <u>(5.6.3.3)</u>

$M_r \ge M_{cr}$ (5.6.3.3)

In which:

$M_{cr} = \gamma_3\gamma_1 S'_{ts} f_r$ (Eq. 5.6.3.3-1)

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

$= 0.24\sqrt{8.5\text{ ksi}}$ (5.4.2.6)

$= 0.70$ ksi

$S'_{ts} = \dfrac{I'}{c_{\text{top of slab}}}$

$= \dfrac{I'}{h_{beam} + t_{fillet} + t_{slab} - C_b}$

$= \dfrac{1343507\text{ in.}^4}{72\text{ in.} + 0\text{ in.} + 8\text{ in.} - 45.08\text{ in.}}$

$= 38474\text{ in.}^3$

$f'_c = 8.5$ ksi

$\gamma_1 = 1.6$

$\gamma_3 = 0.75$

$M_{cr} = 0.75(1.6)(38474\text{ in.}^3)(0.70\text{ ksi})(1\text{ ft.} / 12\text{ in.})$

$= 2693$ k-ft. $< \phi M_n = 7015$ k-ft. OK

Calculation of Stresses for Service and Fatigue Limit States

Because the neutral axis is in the bottom flange, rectangular behavior is observed, and the stresses can be calculated using traditional working stress formulas.

$f_s = \dfrac{M_s}{A_s jd}$

Where:

M_s = applied moment (k-in.)

$M_s = 1.0(601.4\text{ k-ft.}) + 1.0(1107.8\text{ k-ft.}) + 1.0(2588.6\text{ k-ft.})$

$= 4297.8$ k-ft. for factored Service I loading

$M_s = 408.3$ k-ft. for unfactored fatigue truck loading

M_s = 601.4 k -ft. + 1107.8 k-ft.

= 1709.2 k-ft. for DC2 + DW loading, used in fatigue analysis

A_s = 21.33 in.^{2}

b = 38 in.

d = 75.42 in.

$$\rho = \frac{A_s}{bd}$$

$$\rho = \frac{21.33\text{ in.}^2}{(38\text{ in.})(75.42\text{ in.})}$$

= 0.00744

E_s = 29000 ksi

$E_{c,beam}$ = 5362 ksi

$$n = \frac{E_s}{E_{c,beam}}$$

$$= \frac{29000\text{ ksi}}{5362\text{ ksi}}$$

= 5.41

$$k = \sqrt{(0.00744(5.41))^2 + 2(0.00744)(5.41)} - (0.00744)(5.41)$$

= 0.246

$$j = 1 - \frac{k}{3}$$

$$= 1 - \frac{0.246}{3}$$

= 0.92

$$f_s = \frac{4297.8\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(21.33\text{ in.}^2)(0.92)(75.42\text{ in.})}$$

= 35.22 ksi for factored Service I loading

$$f_s = \frac{408.3\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(21.33\text{ in.}^2)(0.91)(75.42\text{ in.})}$$

= 3.34 ksi for Δf in unfactored fatigue truck loading

$$f_s = \frac{1709.2\text{ k}-\text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{(21.33\text{ in.}^2)(0.91)(75.42\text{ in.})}$$

= 14.01 ksi for f_{min} in unfactored fatigue truck loading

<u>Control of Cracking by Distribution of Reinforcement</u> <u>(5.6.7)</u>

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(Eq. 5.6.7-1)}$$

In which:

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

Where:

s = 6 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 = 0.75 for Class 2 exposure

d_c = 2.25 clear + 0.625 in. transverse bar + 0.5(1.0 in. bar)

= 3.375 in.

f_{ss} = 35.22 ksi

h = 80.0 in.

$$\beta_s = 1 + \frac{3.375\text{ in.}}{0.7(80.0\text{ in.}-3.375\text{ in.})}$$

= 1.063

$$s \le \frac{700(0.75)}{(1.063)(35.22\text{ ksi})} - 2(3.375\text{ in.}) = 7.27\text{ in.}$$

6 in. < 7.27 in. OK

Fatigue of Reinforcement (5.5.3.2)

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

In which:

$$(\Delta F)_{TH} = 26 - \frac{22f_{min}}{f_y}$$ (Eq. 5.5.3.2-1)

Where:

$\Delta f = f_{fat,max} - f_{fat,min}$

$= 3.34 \text{ ksi} - 0 \text{ ksi}$

$= 3.34 \text{ ksi}$

$f_{min} = 14.01 \text{ ksi}$ (DC2 + DW stresses)

$\gamma = 1.75$

$$(\Delta F)_{TH} = 26 - \frac{22(14.01\text{ ksi})}{(60\text{ ksi})}$$

$= 20.86 \text{ ksi}$

$\gamma(\Delta f) = 1.75(3.34 \text{ ksi})$

$= 5.85 \text{ ksi} < 20.86 \text{ ksi}$ OK

Service Limit State Stresses (5.9.2.3.2)

Service Stress Limits for Concrete after Losses (5.9.2.3.2)

Compression (For Service I load combination):

$0.60\phi_w f'_c = 0.60(1.0)(8.5\text{ ksi}) = 5.10 \text{ ksi}$ (a)

$0.45f'_c = 0.45(8.5 \text{ ksi}) = 3.83 \text{ ksi}$ (b)

Service stresses are calculated from the following equations:

@ Transfer point from pier:

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

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

Where:

F_S = total prestressing force after all losses (kips)

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

A_{ps} = 5.97 in.^{2} (See App. B)

f_{pbt} = 202.3 ksi

Δf_{pT}= 40.97 ksi

F_S = (5.97 in.^{2})(202.3 ksi - 40.97 ksi)

= 963.1 k

A = 980 in.^{2}

M_{DC1}= 482.3 k-ft.

M_{DC2}= -536.4 k-ft.

M_{DW2}= -988.1 k-ft.

M_{LL+IM}=-2335.9 k-ft.

e = 17.22 in. (See App. B)

S_{bc} = cracked section modulus to bottom of beam (in.^{3}). Because the neutral axis is in the bottom flange, rectangular behavior is observed and working stress equations may be used. Otherwise, the straight-line procedure outlined earlier in the design guide is required.

$$= \frac{I_{cr}}{c_s}$$

I_{cr} = cracked moment of inertia of composite section (in.^{4})

$$= \frac{bc_s^{\,3}}{3} + nA_s\left(d - c_s\right)^2$$

$$c_s = n\rho d\left(-1 + \sqrt{1 + \frac{2}{n\rho}}\right)$$

$$n = \frac{E_s}{E_{c,beam}}$$

$$= \frac{29000\text{ ksi}}{5362\text{ ksi}}$$

$$= 5.4$$

$$\rho = \frac{A_s}{bd}$$

$$= \frac{21.33\text{ in.}^2}{(38\text{ in.})(75.42\text{ in.})}$$

$$= 0.0074$$

$$d = 75.42\text{ in.}$$

$$c_s = (5.4)(0.0074)(75.42\text{ in.})\left(-1+\sqrt{1+\frac{2}{(5.4)(0.0074)}}\right)$$

$$= 18.52\text{ in.}$$

$$I_{cr} = \frac{(38\text{ in.})(18.52\text{ in.})^3}{3} + 5.4\left(21.33\text{ in.}^2\right)(75.42\text{ in.} - 18.52\text{ in.})^2$$

$$= 453375\text{ in.}^4$$

$$S_{bc} = \frac{453375\text{ in.}^4}{18.52\text{ in.}}$$

$$= 24480\text{ in.}^3$$

$$f_b = \frac{963.1\text{k}}{980\text{ in.}^2} + \frac{(963.1\text{k})(17.22\text{ in.})}{24480\text{ in.}^3} - \frac{(482.3\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{24480\text{ in.}^3}$$

$$- \frac{(-536.4\text{ k}-\text{ft.} + -988.1\text{k}-\text{ft.} + -1891.3\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{24480\text{ in.}^3} \qquad \text{(a)}$$

$$= 3.10\text{ ksi} < 5.10\text{ ksi} \qquad \text{OK}$$

$$f_b = \frac{963.1\text{k}}{980\text{ in.}^2} + \frac{(963.1\text{k})(17.22\text{ in.})}{24480\text{ in.}^3} - \frac{(482.3\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{24480\text{ in.}^3}$$

$$- \frac{(-536.4\text{ k}-\text{ft.} + -988.1\text{k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{24480\text{ in.}^3} \qquad \text{(b)}$$

$$= 2.17\text{ ksi} < 3.83\text{ ksi} \qquad \text{OK}$$

Fatigue Limit State Stresses (5.5.3.1)

Fatigue stresses after losses (5.5.3.1)

Compression (For Fatigue I load combination):

$0.40f'_c$ = 0.40(8.5 ksi) = 3.4 ksi

M_{LL+IM} = -380.5 k-ft.

$$f_b = 0.5\left[\frac{F_S}{A}+\frac{F_S e}{S_{bc}}-\frac{M_{DC1}}{S_{bc}}-\frac{(M_{DC2}+M_{DW})}{S_{bc}}\right]-1.75\frac{M_{FL+IM}}{S_{bc}}$$

$$= 0.5\left[\frac{963.1\,k}{980\,in.^2}+\frac{(963.1\,k)(17.22\,in.)}{24480\,in.^3}-\frac{479.1\,k-ft.\left(\frac{12\,in.}{ft.}\right)}{24480\,in.^3}\right]$$

$$-0.5\frac{(-536.4\,k-ft.+-988.1\,k-ft.)\left(\frac{12\,in.}{ft.}\right)}{24480\,in.^3}-1.75\frac{-380.5\,k-ft.\left(\frac{12\,in.}{ft.}\right)}{24480\,in.^3}$$

= 1.46 ksi > -0.55 ksi OK

Check Cutoff Points

The flexural capacity, minimum reinforcement, control of cracking, and fatigue of reinforcement checks are required for the remaining reinforcement at the location where the cutoff reinforcement is terminated. Because those checks are performed in the same manner as those shown above, full calculations will not be shown.

Summary of reinforcement design at reinforcement cutoff point (0.8L):

Flexural Capacity = 3691 k-ft.

Strength I Moment = -2124 k-ft. OK

Cracking Moment OK by inspection (no tension in reinforcement)

Control of Cracking Required Reinforcement Spacing	=	16.49 in.	
Applied Reinforcement Spacing	=	12 in.	OK

Fatigue of Reinforcement Stress Range	=	16.68 ksi	
Allowable Fatigue Stress Range	=	24.46 ksi	OK

Camber and Deflection

<u>Initial Resultant Camber</u>

Camber = $1.80D_{cp} - 1.85D_{cb}$

In which:

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

F_t = 2123 k

L_{beam} = 159.25 ft.

E_{ci} = 4777 ksi

I = 624180 in.^{4}

e_{center} = 22.81 in. (See App. B)

e_{end} = 16.98 in.

$$= \frac{(2123\text{ k})\left(159.25\text{ ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)\right)^2}{(4777\text{ ksi})(624180\text{ in.}^4)}\left[0.0983(22.81\text{ in.}) + 0.0267(16.98\text{ in.})\right]$$

= 7.01 in. up

$$D_{cb} = \frac{5w_{beam}L_{span}^{\ 4}}{384E_{ci}I}$$

w_{beam} = 1.021 k / ft.

L_{span} = 158.0 ft.

E_{ci} = 4777 ksi

$I = 624180 \text{ in.}^4$

$$= \frac{5\left(1.021\text{ k / ft.}\right)\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)\left(158.0\text{ ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)\right)^4}{384\left(4777\text{ ksi}\right)\left(624180\text{ in.}^4\right)}$$

= 4.80 in. down

Camber = 1.80(7.01 in.) - 1.85(4.80 in.)

= 3.73 in. up

<u>Final Resultant Camber for Computing Bearing Seat Elevations</u>

Camber = $1.80D_{cp} – 1.85D_{cb} - D_{cs}$

In which:

$$D_{cs} = \frac{5w_{slab+fillet}L_{span}^{\ 4}}{384E_{ci}I}$$

$w_{slab+fillet}$ = 0.738 k / ft.

L_{span} = 158.0 ft.

E_{ci} = 4777 ksi

$I = 624180 \text{ in.}^4$

$$= \frac{5\left(0.738\text{ k / ft.}\right)\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)\left(158.0\text{ ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)\right)^4}{384\left(4777\text{ ksi}\right)\left(624180\text{ in.}^4\right)}$$

= 2.93 in. down

Camber = 1.80(7.01 in.) - 1.85(4.80 in.) – 2.93 in.

= 0.80 in. up

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

@0.25 point = $0.7125D_{cs}$ = 0.7125(2.93 in.) = 2.09 in.

@0.50 point = D_{cs} = 2.93 in.

@0.75 point = $0.7125D_{cs}$ = 0.7125(2.93 in.) = 2.09 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 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 + 1.75 ft. (face of support to CL pier)

= $0.72(72\text{ in. beam} + 0\text{ in fillet} + 8\text{ in. slab})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) + 1.75\text{ ft.}$

= 6.55 ft. from CL pier

= 6.01 ft. from end of beam

Applied Moments and Shears at Critical Section

M_{DC1} = 884.9 k-ft.
M_{DC2} = -481.7 k-ft.
M_{DW} = -887.3 k-ft.
M_{LL+IM} = -1749.9 k-ft.

$M_{STR\ 1}$ = 1.25(884.9 k-ft. – 481.7 k-ft.) + 1.5(-887.3 k-ft.) + 1.75(-1749.9 k-ft.)
= -3889.3 k-ft.

V_{DC1} = -129.3 k-ft.
V_{DC2} = -17.7 k-ft.
V_{DW} = -32.5 k-ft.
V_{LL+IM} = -111.2 k-ft.

$V_{STR\ 1}$ = 1.25(-129.3 k-ft. – 17.7 k-ft.) + 1.5(-32.5 k-ft.) + 1.75(-111.2 k-ft.)
= -427.1 k-ft.

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:

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

Where:

d_e = 75.42 in. (see d_s in moment calculations)

a = 4.66 in. (see moment calculations)

$$d_v = 75.42 \text{ in.} - \left(\frac{4.66 \text{ in.}}{2}\right) \qquad \text{(C5.8.2.9)}$$

$= 73.09$ in.

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

$0.9d_e$ = 0.9(74.59 in.)

= 67.13 in.

$0.72h$ = 0.72(81.5 in.)

= 58.68 in.

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

d_v = 73.09 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} (5.9.4.3.1)

Transfer Length $= 60d_b$ (5.9.4.3.1)

$= 60(0.6 \text{ in.})$

$= 36 \text{ in.}$

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

$l_d = 144.3$, calculated previously

The critical section is at $x = 6.01 \text{ ft.} \left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 72.1 \text{ in.}$

36 in. < 72.1 in. < 144.3 in. Therefore,

$$f_{px} = f_{pe} + \frac{l_{px} - 60d_b}{l_d - 60d_b}\left(f_{ps} - f_{pe}\right) \quad \text{(Eq. 5.9.4.3.2-3)}$$

Where:

$f_{ps} = 258.0 \text{ ksi}$

$f_{pe} = 161.23 \text{ ksi}$

$l_{px} = 73.7 \text{ in.}$

$l_d = 144.5 \text{ in.}$

$d_b = 0.6 \text{ in.}$

$$f_{px} = 161.23 \text{ ksi} + \frac{72.1 \text{ in.} - 60(0.6 \text{ in.})}{144.3 \text{ in.} - 60(0.6 \text{ in.})}(258.0 \text{ ksi} - 161.23 \text{ ksi}) \quad \text{(Eq. 5.11.4.2-3)}$$

$= 193.5 \text{ ksi}$

% development = 193.5 ksi / 258 ksi = 75%

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

$= \left(\frac{0.217\text{ in.}^2}{\text{strand}}\right)(4\text{ harped strands})$

$= 0.868\text{ in.}^2$

$\Psi = 4.26^\circ$

$V_p = (0.868\text{ in.}^2)(193.5\text{ ksi})(\sin 4.26^\circ)$

$= 12.47\text{ k}$

Shear Resistance Due to Concrete V_c (5.7.3.3)

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

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v \quad \text{(Eq. 5.7.3.3-3)}$$

Where:

$f'_c = 8.5\text{ ksi}$

$d_v = 73.09\text{ in.}$

$b_v = 7\text{ in.}$

$\beta = \frac{4.8}{1+750\varepsilon_s}$

Where:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{\left(E_sA_s + E_pA_{ps}\right)} \geq 0 \quad \text{(Eq. 5.7.3.4.2-4)}$$

Where:

$|M_u| = 46671\text{ k-in.} > |V_u - V_p|d_v = 30304\text{ k-in.}$

$N_u = 0\text{ k}$

$V_u = -427.1\text{ k}$

$V_p = 12.47\text{ k}$

A_{ps} = (0.217 in.^{2})(6 strands in flexural tension side)(75% development)

= 0.98 in.^{2}

f_{po} = 0.7(270 ksi)

= 189 ksi

E_s = 29000 ksi

A_s = 21.33 in.^{2}

E_p = 28500 ksi

$$\varepsilon_s = \frac{\left(\frac{|46671\text{ k}-\text{in.}|}{73.09\text{ in.}} + 0.5(0\text{ k}) + |-427.1\text{ k} - 12.47\text{ k}| - (0.98\text{ in.}^2)(189\text{ ksi})\right)}{\left((29000\text{ ksi})(21.33\text{ in.}^2) + (28500\text{ ksi})(0.98\text{ in.}^2)\right)}$$

= 0.00135

$$\beta = \frac{4.8}{1+750(0.00135)}$$

= 2.39

$$V_c = 0.0316(2.39)\sqrt{8.5\text{ ksi}}(7\text{ in.})(73.09\text{ in.})$$

= 112.7 k

Required Spacing of Transverse Reinforcement for Nominal Shear Resistance

As defined earlier, solving Equations 5.7.3.3-1, 5.7.3.3-3, C5.7.3.3-1, and 5.7.2.1-1 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 = 2(0.31 in.^{2})

= 0.62 in.^{2}

f_y = 60 ksi

d_v = 73.09 in.

θ = 29 + 3500(0.00135) (Eq. 5.8.3.4.2-3)

= 33.72 degrees

V_u = 427.1 k

ϕ = 0.9

V_p = 12.47 k

V_c = 112.7 k

$$s \le \frac{(0.62\text{ in.}^2)(60\text{ ksi})(73.09\text{ in.})\cot 33.27^\circ}{\frac{427.1\text{k}}{0.9} - 12.47\text{ k} - 112.7\text{ k}}$$

$\le$ 11.5 in.

<u>Maximum Permitted Spacing of Transverse Reinforcement</u> (5.7.2.6)

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

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

s_{max}= $0.8d_v \le 24.0\text{ in.}$ (Eq. 5.7.2.6-1)

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

s_{max}= $0.4d_v \le 12.0\text{ in.}$ (Eq. 5.7.2.6-2)

Where:

f'_c = 8.5 ksi

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

$$= \frac{|427.1\text{k} - 0.9(12.47\text{ k})|}{0.9(7\text{ in.})(73.09\text{ in.})}$$ (Eq. 5.7.2.8-1)

= 0.90 ksi

$0.125f'_c$ = 0.125(8.5 ksi) = 1.06 ksi > 0.90 ksi

Therefore, $s_{max} = 0.8d_v \leq 24.0$ in.

$0.8d_v$ = 0.8(73.09 in.) = 58.5 in. > 24 in.

Use 24 in. for maximum allowable spacing when determining final spacing at critical section

Minimum Transverse Reinforcement (5.7.2.5)

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 = 0.62 in.^{2}

f_y = 60 ksi

f'_c = 8.5 ksi

b_v = 7 in.

$$s = \frac{\left(0.62\text{ in.}^2\right)\left(60\text{ ksi}\right)}{0.0316\sqrt{8.5\text{ ksi}}\left(7\text{ in.}\right)}$$

= 57.7 in.

Interface Shear Transfer Reinforcement (5.7.4)

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

Where:

c = 0.28

A_{cv} = (24 in.)(12 in.) = 288 in.^{2}

μ = 1.0

A_{vf} = 0.62 in.^{2}

f_y = 60 ksi

P_c = 0 k

K_1 = 0.3

K_2 = 1.8 ksi

$v_{ui} = \dfrac{V_u}{b_{vi}d_v}$ (Eq. 5.7.4.5-1)

$= \dfrac{427.1\text{k}}{(24\text{ in.})(73.09\text{ in.})}$

= 0.24 ksi

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

V_{ui} = (0.24 ksi)(288 in.^{2})

= 69.12 k

$\dfrac{V_{ui}}{\phi} = \dfrac{69.12\text{ k}}{0.9}$

= 76.8 k

$s = \dfrac{12(0.6)(0.62\text{ in.}^2)(60\text{ ksi})}{76.8\text{ k} - 0.28(288\text{ in.}^2)}$

< 0, the concrete friction alone is enough to transfer the interface shear. Check the minimum reinforcement requirements.

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

76.8 k $\leq$ (0.2)(8.5 ksi)(288 in.^{2}) = 489.6 k OK

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

76.8 k $\leq$ (0.8)(288 in.^{2}) = 230.4 k OK

Per Article 5.7.4.2, the minimum required area of interface shear reinforcement is:

$$A_{vf} = \frac{0.05A_{cv}}{f_y} \quad \text{(Eq. 5.7.4.2-1)}$$

$$= \frac{0.05\left(288\text{ in.}^2\right)}{60\text{ ksi}}$$

$$= 0.24\text{ in.}^2$$

s = (0.62 in.^{2})(12 in. / 0.24 in.^{2}) = 31 in.

Determine Controlling Transverse Reinforcement Spacing

The required reinforcement spacing for strength is 11.5 in.
The maximum permitted reinforcement spacing is 24 in.
The minimum reinforcement area is based on a spacing of 57.7 in.
The required reinforcement for interface shear is 31 in.

The controlling spacing at the critical section for shear is 11.5 in. Because standard WWR reinforcement spacings are 3 in., 6 in., 12 in., and 24 in., use 6 in. spacing.

Longitudinal Reinforcement (5.7.3.5)

$$A_s f_y + A_{ps} f_{ps} \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta \quad \text{(Eq. 5.7.3.5-1)}$$

Where:

A_s = 21.33 in.^{2}
f_y = 60 ksi
A_{ps} = 0.98 in.^{2} effective strands on flexural tension side of member
f_{ps} = 189 ksi
$|M_u|$ = 46671 k-in.
d_v = 73.09 in.
ϕ_f = 0.9

$|V_u|$ = 427.1 k

ϕ_v = 0.9, assuming non-prestressed concrete

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad \text{(Eq. 5.7.3.3-2)}$$

$$= \frac{(0.62\text{ in.}^2)(60\text{ ksi})(73.09\text{ in.})\cot 33.72°}{6\text{ in.}}$$

$= 678.9\text{ k} > V_u / \phi_v = 474.6\text{ k}$, use 474.6 k

V_p = 12.47 k

θ = 33.72 degrees

$A_s f_y + A_{ps} f_{ps}$ = $(21.33\text{ in.}^2)(60\text{ ksi}) + (0.98\text{ in.}^2)(189\text{ ksi})$
= 1465.02 k

$$\frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta =$$

$$\frac{|46671\text{ k}-\text{in.}|}{(73.09\text{ in.})(0.9)} + 0.5\frac{0\text{ k}}{0.9} + \left(\frac{|474.6\text{ k}|}{0.9} - 0.5(474.6\text{ k}) \; -12.47\text{ k}\right)\cot 33.72°$$

= 1124.8 k

1465.2 k > 1124.8 k OK

<u>Check Final Design Against Eq. 5.7.3.3-2</u>

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

$$V_n = 0.25f'_c b_v d_v + V_p \qquad \text{(Eq. 5.7.3.3-2)}$$

Where:

f'_c = 8.5 ksi

b_v = 7 in.

d_v = 73.09 in.

V_p = 12.47 k

V_n = 0.25(8.5 ksi)(7 in.)(73.09 in.) + 12.47 k
= 1099.7 k

$V_c + V_s + V_p$ = 112.7 k + 678.9 k + 12.47 k = 804.07 k OK

Tabulated Required Maximum Spacings

All spacings are in inches. Span 2 is similar by symmetry.

Pt.	Strength	Max Spacing	Min Reinf.	Horizontal Shear	Controlling
Critical Section	21.8	24.0	57.1	24.0	21.8
0.1	30.4	24.0	57.1	24.0	24.0
0.2	75.9	24.0	57.1	24.0	24.0
0.3	126.1	24.0	57.1	24.0	24.0
0.4	*	24.0	57.1	24.0	24.0
0.5	815.1	24.0	57.1	24.0	24.0
0.6	72.7	24.0	57.1	24.0	24.0
0.7	48.9	24.0	57.1	24.0	24.0
0.8	25.4	24.0	57.1	24.0	24.0
0.9	17.2	24.0	57.1	24.0	17.2
Critical Section	11.5	24.0	57.1	24.0	11.5

*At 0.4L, the applied shears are near zero, resulting in reinforcement spacing for strength to be very, very large. These shears obviously do not control the spacing of the final design.

Shear spacings are standardized to 3 in., 6 in., 12 in., and 24 in. Symmetry is also encouraged. Therefore, use 6 in. spacing for the last 10%. of each beam end, 12 in. spacing for the next 10%, and 24 in. spacing in the interior.

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	Crit. Sec										Crit. Sec	
Moment	0.0428	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	0.9572	1
DC1	905.6	2004.3	3563.1	4676.6	5344.7	5567.4	5344.7	4676.6	3563.1	2004.3	1147.9	0
DC2	72.3	156.4	264.6	324.7	336.8	300.7	216.5	84.2	-96.2	-324.7	-446	-601.4
DW	133.1	288	487.4	598.2	620.4	553.9	398.8	155.1	-177.2	-598.2	-819	-1107.8
LL+IM+	500.2	1087	1861.6	2337.9	2548.7	2505.2	2230.8	1728.3	1031.2	344.1	153.8	0
LL+IM-	-61.2	-144.2	-288.3	-432.5	-576.7	-720.8	-865	-1009.2	-1153.3	-1460.3	-1694.9	-2595.3
LL+IM (Fatigue max)	150.7	324.1	543.5	681.7	728.5	710.7	643.7	506.6	315.3	88	0	0
LL+IM (Fatigue min)	-17.1	-40.2	-80.5	-120.7	-161	-201.2	-241.5	-281.7	-321.9	-362.2	-380.5	-402.4

Shear	0.0428	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	0.957	1
DC1	128.1	112	84	56	28	0	-28	-56	-84	-112	-124.7	-140
DC2	10.1	8.3	5.3	2.3	-0.8	-3.8	-6.8	-9.8	-12.8	-15.9	-18.2	-18.9
DW	18.5	15.9	9.7	4.2	-1.4	-7	-12.5	-18.1	-23.7	-29.2	-31.8	-34.8
LL+IM	92.2	83.2	68.4	54.8	42.4	-53.9	-66.4	-79	-91.6	-104	-109.5	-116

APPENDIX B: Strand Group Eccentricity Calculations

Angle of Inclination of Harped Strands = $\arctan\left(\frac{\text{harped strand drop in height}}{0.4 \text{ * beam length}}\right)$

The harped strands are in two rows. At the ends of the beams, the harped strands are in rows 2T and 3T, with heights of 67 in. and 65 in. above the bottom of the beam, respectively. At the harping point, the harped strands are in rows 5 and 4, with heights of 10 in. and 8 in. above the bottom of the beam. Therefore:

Harped strand drop in height = 67 in. – 10 in. = 57 in.

$$0.4 \text{ * beam length} = 0.4(159.25 \text{ ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)$$

$$= 764.4 \text{ in.}$$

$$\text{Angle of inclination } \Psi = \arctan\left(\frac{57 \text{ in.}}{764.4 \text{ in.}}\right)$$

$$= 4.26^{o}$$

Transfer Length (36 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	2.68	62.32
5B	67	Y	2.68	64.32
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	62.5	8.75	4	0	0				8.75
2B	14	62.5	8.75				4	0	0	8.75
3B	10	62.5	6.25							6.25
4B	2	62.5	1.25							1.25
5B	2	62.5	1.25							1.25
1T	2	62.5	1.25							1.25
									Σ =	27.5

A_{ps} = 0.217 in.^{2} / strand * 27.5 eff. strands = 5.97 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	8.75	17.5
2B	4	8.75	35
3B	6	6.25	37.5
4B	62.32	1.25	77.9
5B	64.32	1.25	80.4
1T	69	1.25	86.25
		Σ =	334.55

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,\text{Dev.})\right)}{\sum \Sigma(N * \%\,\text{Dev.})} = 29.39\text{ in.} - \frac{334.55\text{ in.}}{27.5} = 17.22\text{ in.}$$

Critical Section for Shear (59.34 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	4.42	60.58
5B	67	Y	4.42	62.58
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	70.54	9.88	4	0	0				9.88
2B	14	70.54	9.88				4	0	0	9.88
3B	10	70.54	7.05							7.05
4B	2	70.54	1.41							1.41
5B	2	70.54	1.41							1.41
1T	2	70.54	1.41							1.41
									Σ =	31.04

A_{ps} = 0.217 in.^{2} / strand * 31.04 eff. strands = 6.74 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	9.88	19.76
2B	4	9.88	39.52
3B	6	7.05	42.3
4B	60.58	1.41	85.4178
5B	62.58	1.41	88.2378
1T	69	1.41	97.29
		Σ =	372.5256

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,Dev.)\right)}{\sum \Sigma(N * \%\,Dev.)} = 29.39\text{ in.} - \frac{372.53\text{ in.}}{31.04} = 17.39\text{ in.}$$

Centroid of Lifting Loops (144 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	10.74	54.26
5B	67	Y	10.74	56.26
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	99.71	13.96	4	0	0				13.96
2B	14	99.71	13.96				4	0	0	13.96
3B	10	99.71	9.97							9.97
4B	2	99.71	1.99							1.99
5B	2	99.71	1.99							1.99
1T	2	99.71	1.99							1.99
									Σ =	43.86

A_{ps} = 0.217 in.^{2} / strand * 43.86 eff. strands = 9.52 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	13.96	27.92
2B	4	13.96	55.84
3B	6	9.97	59.82
4B	54.26	1.99	107.9774
5B	56.26	1.99	111.9574
1T	69	1.99	137.31
		Σ =	500.8248

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,\text{Dev.})\right)}{\sum \Sigma(N * \%\,\text{Dev.})} = 29.39\text{ in.} - \frac{500.82\text{ in.}}{43.86} = 17.97\text{ in.}$$

Debond Location 1 (240 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	17.9	47.1
5B	67	Y	17.9	49.1
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	100	14	4	0	0				14
2B	14	100	14				4	0	0	14
3B	10	100	10							10
4B	2	100	2							2
5B	2	100	2							2
1T	2	100	2							2
									Σ =	44

A_{ps} = 0.217 in.^{2} / strand * 44 eff. strands = 9.55 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	14	28
2B	4	14	56
3B	6	10	60
4B	47.1	2	94.2
5B	49.1	2	98.2
1T	69	2	138
		Σ =	474.4

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,Dev.)\right)}{\sum \Sigma(N * \%\,Dev.)} = 29.39\text{ in.} - \frac{474.4\text{ in.}}{44} = 18.61\text{ in.}$$

Debond Location 1 + Transfer/Debond Location 2 (276 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	20.58	44.42
5B	67	Y	20.58	46.42
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	100	14	4	62.5	2.5				16.5
2B	14	100	14				4	0	0	14
3B	10	100	10							10
4B	2	100	2							2
5B	2	100	2							2
1T	2	100	2							2
									Σ =	46.5

A_{ps} = 0.217 in.^{2} / strand * 46.5 eff. strands = 10.10 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	16.5	33
2B	4	14	56
3B	6	10	60
4B	44.42	2	88.84
5B	46.42	2	92.84
1T	69	2	138
		Σ =	468.68

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,Dev.)\right)}{\sum \Sigma(N * \%\,Dev.)} = 29.39\text{ in.} - \frac{468.68\text{ in.}}{46.5} = 19.31\text{ in.}$$

Debond Location 2 + Transfer (312 in. from beam end)

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	23.27	41.73
5B	67	Y	23.27	43.73
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	100	14	4	71.84	2.8736				16.8736
2B	14	100	14				4	62.5	2.5	16.5
3B	10	100	10							10
4B	2	100	2							2
5B	2	100	2							2
1T	2	100	2							2
									Σ =	49.3736

A_{ps} = 0.217 in.^{2} / strand * 49.37 eff. strands = 10.71 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	16.8736	33.7472
2B	4	16.5	66
3B	6	10	60
4B	41.73	2	83.46
5B	43.73	2	87.46
1T	69	2	138
		Σ =	468.6672

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,\text{Dev.})\right)}{\sum \Sigma(N * \%\,\text{Dev.})} = 29.39\text{ in.} - \frac{468.67\text{ in.}}{49.37} = 19.90\text{ in.}$$

<u>0.4L, 0.5L, Point of Maximum Moment</u>

Determine Row Heights:

Row	Strand Height @ Beam End (in.)	Harped?	Harped Strand Drop (in.)	Strand Height m @ Transfer (in.)
1B	2	N		2
2B	4	N		4
3B	6	N		6
4B	65	Y	57	8
5B	67	Y	57	10
1T	69	N		69

Determine % Development of Strands

	Strands Debonded @ Beam End			Strands Debonded @ Debond Location 1			Strands Debonded @ Debond Location 2			
	# Strands		N *	# Strands		N *	# Strands		N *	
Row	N	% Dev.	% Dev.	N	% Dev.	% Dev.	N	% Dev.	% Dev.	Σ(N * % Dev.)
1B	14	100	14	4	100	4				18
2B	14	100	14				4	100	4	18
3B	10	100	10							10
4B	2	100	2							2
5B	2	100	2							2
1T	2	100	2							2
									Σ =	52

A_{ps} = 0.217 in.^{2} / strand * 52 eff. strands = 11.28 in.^{2}

Determine Centroid of Strand Group

Row	m	Σ(N * % Dev.)	m * Σ(N * % Dev.)
1B	2	18	36
2B	4	18	72
3B	6	10	60
4B	8	2	16
5B	10	2	20
1T	69	2	138
		Σ =	342

$$\text{Eccentricity} = C_b - \frac{\sum\left(m * \Sigma(N * \%\,\text{Dev.})\right)}{\sum \Sigma(N * \%\,\text{Dev.})} = 29.39\text{ in.} - \frac{342\text{ in.}}{52} = 22.81\text{ in.}$$