14. Structural Concrete

Section 5 of the *LRFD Specifications* presents unified design requirements for concrete, both reinforced and prestressed, in all structural elements. This chapter presents DOT&PF supplementary information specifically on the properties of concrete, reinforcing steel, and prestressing strands and the design of structural concrete members.

14.1. Materials

14.1.1. Structural Concrete

Reference: LRFD Article 5.4.2

Table 14-1 presents DOT&PF criteria for the minimum 28-day compressive strength of concrete in structural elements. Normal weight concrete varies between 145 pcf (for cast-in-place concrete) and 155 pcf (for precast concrete) excluding the weight of the internal steel reinforcement.

14.1.2. Reinforcing Steel

Reference: LRFD Article 5.4.3

For both general and seismic applications, reinforcing steel must conform to the requirements of ASTM A706, Grade 60. (A706 is available as either Grade 60 or Grade 80, but Grade 80 is not permitted in members expected to form plastic hinges such as columns.) Use 29,000 ksi as the modulus of elasticity, E_s. ASTM A706 reinforcing steel is manufactured with controlled material properties. These properties include a maximum yield strength, a minimum ratio between the tensile and yield strengths, and a higher rupture strain than conventional ASTM A615 reinforcement. In addition, ASTM A706 reinforcing steel is manufactured with a controlled chemical composition making it more weldable. Comply with *Alaska Standard Specifications* Section 503 for welding of reinforced steel.

The bridge engineer may substitute ASTM A615 only with the approval of the Chief Bridge Engineer.

Seismic Modeling of Reinforcing Steel

When designing reinforced concrete members that are expected to form plastic hinges (i.e., earthquake-resisting elements) according to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* use the steel reinforcement properties in Table 14-2. The code provisions are based primarily on preserving life safety through a "no collapse" design objective. Some bridges may be required to perform to a higher design objective following a seismic event.

Bridges that suffer minimal damage may be expected to open shortly after the design seismic event provided that the strain in the concrete compression face not exceed 0.004 and the strain in the extreme longitudinal tension reinforcing does not exceed the onset of strain hardening, ε_{sh}, as shown in Table 14-2.

Circular column damage following the design seismic event is expected to be repairable provided that the longitudinal reinforcing steel does not buckle (Goodnight et. al. 2015). For circular columns satisfying $\rho_l < 0.03$ and $\rho_s > 0.005$, the longitudinal steel strain associated with the onset of bar buckling, ε_s^{bb}, may be calculated as:

$$\varepsilon_s^{bb} = 0.032 + 790\rho_s F_{yhe}/E_s - 0.14P / (f'_{ce}A_g)$$

where:

A_g = gross area of member cross section (in.^{2})

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

A_{st} = total area of column reinforcement (in.^{2})

A_{st} = total area of column reinforcement (in.^{2})

D' = core diameter of column measured from center of spiral or hoop (in.)

E_s = modulus of elasticity of reinforcing steel (ksi)

f'_{ce} = expected concrete compressive strength (ksi)

F_{yhe} = expected strain of transverse reinforcing steel (ksi)

P = unfactored axial dead load on column (k)

S = pitch of spiral or spacing of hoop reinforcement (in.)

ρ_l = longitudinal reinforcement ration in the column
= A_{st} / A_g

ρ_s = volumetric reinforcement ratio
= $4A_{sp} / (s \cdot D')$

The repairable strain limits for non-circular column cross sections have not been developed.

Table 14-1
Compressive Strength of Concrete

Structural Element	Minimum 28-Day Compressive Strength (f'_c)
Bridge Decks (Class AA Concrete)	5 ksi
Prestressed Concrete (Precast) (Class P Concrete)	5 ksi
Prestressed Concrete (Cast-in-Place) (Class A Concrete)	4 ksi
Piers and Columns (Class A Concrete)	4 ksi
Abutments (Class A Concrete)	4 ksi
Wingwalls and Retaining Walls (Class A Concrete)	4 ksi
Spread Footings (Class A Concrete)	4 ksi
Drilled Shafts (Class DS Concrete)	4 ksi
Barrier Rails and Rail Curb, etc. (Class A Concrete)	4 ksi
Miscellaneous (e.g., Culverts, Sound Walls) (Class A Concrete)	4 ksi
Approach Slabs	See 16.3

14.1.3. Prestressing Strands

Reference: LRFD Article 5.4.4

Use low-relaxation, 7-wire prestressing strand with a minimum tensile strength of f_{pu} = 270 ksi and a minimum yield strength of $f_{py} = 0.9f_{pu}$. If more precise data is not available, the modulus of elasticity for prestressing strands, based on nominal cross-sectional area, may be taken as 28,500 ksi.

Table 14-2
Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A706 Grade 60	ASTM A706 Grade 80
Specified minimum yield stress (ksi)	f_y	#3-#18	60	80
Expected yield stress (ksi)	f_{ye}	#3-#18	68	85
Expected tensile strength (ksi)	f_{ue}	#3-#18	95	112
Expected yield strain	ε_{ye}	#3-#18	0.0023	0.0033
Onset of strain hardening	ε_{sh}	#3-#8	0.0150	0.0074
		#9	0.0125	0.0074
		#10 & #11	0.0115	0.0074
		#14	0.0075	0.0074
		#18	0.0050	0.0074
Reduced ultimate tensile strain	ε_{su}^{R}	#4-#10	0.090	0.06
		#11-#18	0.060	0.06
Ultimate tensile strain	ε_{su}	#4-#10	0.120	0.095
		#11-#18	0.090	0.095

14.2. Reinforcement

14.2.1. Reinforcing Steel

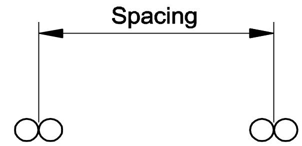
Spacing of Bars

Reference: LRFD Article 5.10.3

Table 14-3 presents DOT&PF criteria for minimum center-to-center spacing between reinforcement bars based on bar size and spliced vs unspliced. The accompanying sketch illustrates how to measure the spacing for spliced bars. Epoxy-coated bars spaced at less than six bar diameters, which includes the majority of the table, require an additional increase in development lengths over wider spaced epoxy-coated bars.

Use calculations and large-scale drawings to carefully check fit and clearance of reinforcing. Skews will tend to complicate problems with reinforcing fit. Consider tolerances normally allowed for cutting, bending, and locating reinforcing. Refer to the *Alaska Standard Specifications for Highway Construction* for tolerances.

Table 14-3
Minimum Spacing of Bars

Bar Size	Minimum Spacing	
	Unspliced Bars	Spliced Bars (assumes a side-by-side lap) 
#4	3″	3½″
#5	3″	3½″
#6	3″	4″
#7	3½″	4″
#8	3½″	4½″
#9	3½″	4½″
#10	4″	5″
#11	4″	5½″
#14	4½″	6″
#18	6″	8″

Fabrication Lengths

Use a maximum unspliced length of 60 feet for detailing reinforcing steel. In remote locations, use 40 feet (for shipping).

Lateral Confinement Reinforcement

Reference: LRFD Article 5.11.4.

Detail all lateral column reinforcement according to the requirements in the *Guide Specifications for LRFD Seismic Bridge Design*. Lateral reinforcement for compression members must consist of either spiral reinforcement or electric resistance butt-welded hoops.

Lateral Confinement Reinforcement for Drilled Shafts. The reinforcing steel cage for drilled shafts must extend the full depth of the shaft.

Determine the length of the plastic hinge confinement reinforcement by appropriate analysis, but the length must not be less than the requirements of the *Guide Specifications for LRFD Seismic Bridge Design*.

Maximize the size of longitudinal and transverse reinforcement to increase the openings between all reinforcement to allow concrete to pass through the cage during placement. The window formed between the longitudinal and transverse bars should not be less than 5 inches by 5 inches. Maintain the maximum spacing requirements of LRFD Article 5.11.4.5.2.

Epoxy-Coated Reinforcing Steel

Epoxy-coated reinforcement bars are required at the following locations:

- bridge decks (both layers)
- reinforcing that extends into bridge decks and terminates within 12 inches of the top of the deck slab
- bridge approach slabs
- barrier rails and rail curbs

- sidewalks

Standard End Hook Development Length in Tension

Reference: LRFD Article 5.10.8.2.4.

Closed ties always fold the hooked end at 135° into the core. See Figure 14-1. Make both tails parallel for confinement with the specified length (LRFD Article 5.10.2.2). Standard hooks use a 90-degree and 180-degree bend to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes.

For the development of standard hooks, refer to Figure C5.10.8.2.4a-1 in the *LRFD Specifications* for hooked-bar details. Use the same figure for both uncoated and coated bars.

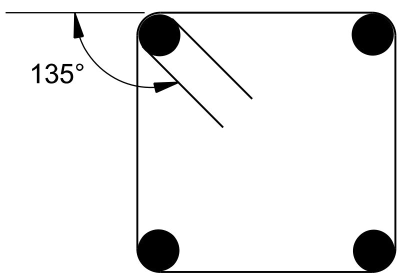


Figure 14-1
End Hook for Closed Ties

Splices

Reference: LRFD Article 5.10.8.4.

1. **Deck Steel over Piers.** Over piers, do not provide any splices in the longitudinal deck reinforcing steel where the tensile stress in the deck is calculated to be greater than the rupture modulus of the concrete. The contract documents must clearly identify these regions. In practice, especially in remote locations, splices may be unavoidable. In these cases, optimize the location of the no-splice zone and indicate it on the plans.

2. **Plastic Hinge Regions.** In columns and drilled shafts, do not provide any splices in the longitudinal reinforcing or splicing of spiral reinforcing within the plastic hinge regions. The contract documents must clearly identify these regions. In practice, especially in remote locations, splices may not be avoidable. In these cases, optimize the location of the no-splice zone and stagger the splices as much as practical.

Spiral Reinforcing Steel Calculation

Use the equation $S = n\,((\pi d)^2 + h^2)^{1/2}$ to calculate the length of spiral reinforcing steel for any spiral segment with a constant pitch, where:

h = pitch of spiral segment
n = number of turns
d = outer bar diameter

To calculate the total length of reinforcing steel in a spiral, add the length of each segment, S, with the length of additional top and bottom turns.

14.2.2. Prestressing Strands for Pretensioned Girders

Strand Size

Common sizes of prestressing strands used in bridge construction are ½-inch and 0.6-inch diameter. However, for girders fabricated within Alaska, the diameter of the prestressing strands in pretensioned girders is ½ inch. For girders fabricated outside of Alaska, the diameter is typically 0.6 inch.

Strand Spacing

The typical strand spacing is 2 inches center-to-center.

Strand Profile

It is acceptable to use either a straight or harped strand profile for precast members. DOT&PF prefers the use of harped strands over the debonding of straight strands. Use a combination of debonded and harped strands when necessary to satisfy design requirements, subject to the following:

- Only use debonded strands when required by analysis.
- Do not debond more than 40 percent of the strands.
- Do not debond more than one-half of the strands in a row.
- Do not debond harped strands.

Harped Strands

In precast, pretensioned girders harped strands are bundled at the harp points and the slope of the harped strands should not exceed 1:7 (9 degrees).

Strand Patterns

Fully detail the strand pattern showing the total number of strands, layout and spacing, edge clearances, which strands will be harped and/or debonded, and the layout of all mild reinforcing steel.

Frequently, it may be preferable to design precast, pretensioned girders of the same size and similar length in the same bridge or within bridges on the same project with a slightly different number of strands. In this case, consider using the same number and pattern of strands (including height of harping) for these girders to facilitate fabrication. Figure 14-2 presents a typical strand pattern.

Strand Splicing

Do not splice prestressing strand.

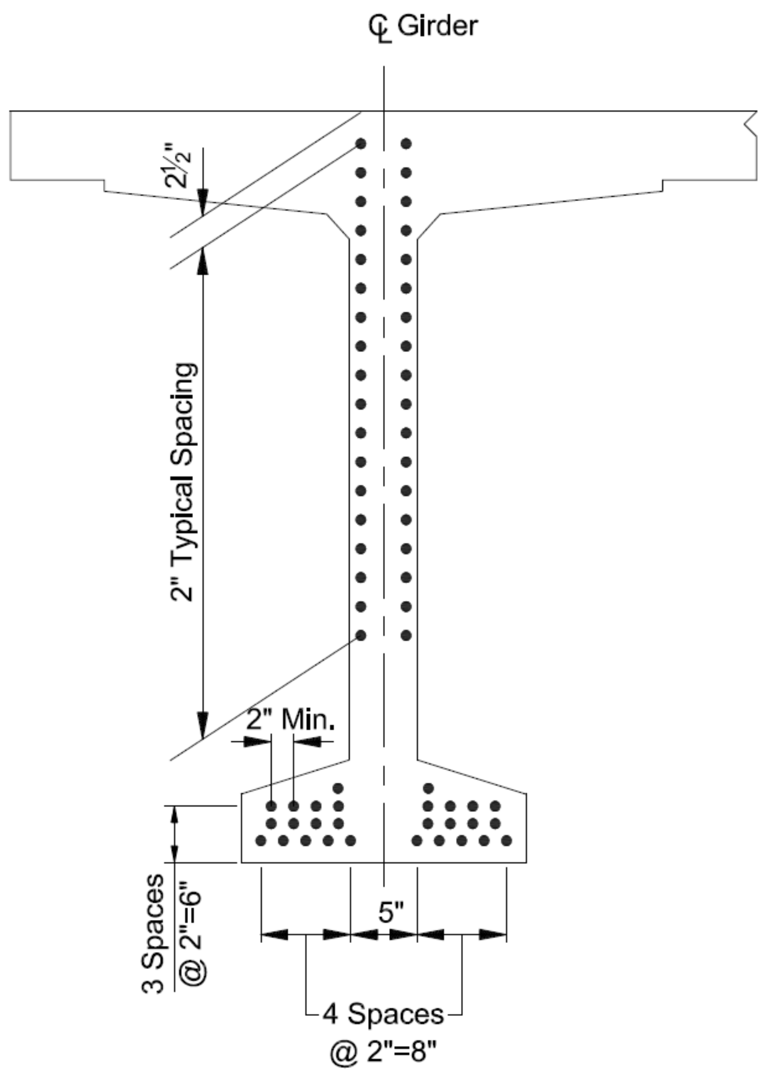


Figure 14-2
Typical Strand Pattern

14.3. Structural Concrete Design

14.3.1. Member Design Models

The *LRFD Specifications* provides two design approaches for concrete members — the traditional sectional design model and the strut-and-tie model. The sectional design model is based upon traditional beam theory wherein planar sections remain plane after loading. The strut-and-tie model may be used where traditional theory is applicable and in disturbed regions where planar sections do not remain planar after loading.

14.3.2. Sectional Design Model

The sectional design model is appropriate for the design of typical bridge girders, slabs, columns, and other regions of components where the assumptions of traditional beam theory are valid. The sectional design model assumes that the response at a particular section depends only on the calculated values of the sectional force effects such as moment, shear, axial load, and torsion. This model does not consider the specific details of how the force effects were introduced into the member. LRFD Article 5.7.3. discusses the sectional design model and describes the applicable geometry required to use this technique to design for shear.

Flexural Resistance

Obtain the flexural resistance of a girder section by using the rectangular stress distribution of LRFD Article 5.6.2.2. In lieu of using this simplified, yet accurate approach, use a strain compatibility approach as outlined in LRFD Article 5.6.3.2.5. Base the general equation for structural concrete flexural resistance of LRFD Article 5.6.3.2.1. upon the rectangular stress block or other constitutive model capable of accurately representing the magnitude and location of the resultant concrete compressive force.

Minimum flexural reinforcement requirements are specified in LRFD. The LRFD requirements are satisfied when either:

(1) The factored nominal moment, ϕM_n, is greater than 1.33 times the factored ultimate moment, M_u. This requirement does not necessarily result in a flexural member with any significant deformation or ductile behavior. Non-ductile members fail without significant warning, or signs of distress prior to fracture. In this situation, the reserve strength provided by the member is deemed adequate to preclude failure of non-ductile members.

(2) The factored nominal moment, ϕM_n, exceeds the expected concrete cracking moment, M_{cr}, defined by Equation 5.6.3.3-1. Flexural members that satisfy this requirement are capable of developing sufficiently large deformations to provide warning of impending failure. That is, members that satisfy this requirement provide for a ductile response.

Design reinforced concrete flexural members to satisfy the ductility requirements of Equation 5.6.3.3-1.

Crack Control Reinforcement

Reference: LRFD Article 5.6.7.

When designing for crack control, use the following values unless a more severe condition is warranted:

- γ_e = 0.75 (Class 2 exposure condition where the assumed crack width equals 0.013 inch) for footings and other components in contact with soil or brackish water, for decks, slabs, barrier rail, tops of abutment caps below expansion joints, and other components susceptible to deicing agent exposure; and
- γ_e = 1.00 (Class 1 exposure condition where the assumed crack width equals 0.017 inch) for all other components.

Several smaller reinforcing bars at moderate spacing are more effective in controlling cracking (more numerous but narrower crack widths) than fewer larger bars (fewer but wider cracks).

Shear Resistance

Provide adequate shear reinforcement in bridge girders so that the shear and moment load-rating factors are approximately equal (within about 20 percent). The zero-tension stress limit of Section 14.4.2 often results in girders with greater flexural resistance than that required in the Strength limit state load combinations. However, no similar provisions exist for shear. To ensure a ductile bridge response mechanism, additional shear reinforcement will be required.

When calculating the net longitudinal strain, i.e. ε_s or ε_x, include the effects of strand development length, ℓ_d, in the E_pA_{ps} denominator term only. Include the prestress transfer length adjustment in the $A_{ps}f_{po}$ numerator term only.

Sectional Design Models. Sectional design models are appropriate for flexural regions, regions away from reactions, applied loads, and changes in cross section, where conventional methods for the strength of materials are applicable and strains are linear. The LRFD Specifications present two alternative sectional shear design models for estimating the shear resistance of concrete members:

1. **Simplified Procedure for Nonprestressed Sections**. (Reference: LRFD Article 5.7.3.4.1). This procedure may be used for conventionally reinforced concrete members without axial loads that are less than 16 inches. The shear resistance determined by this procedure is essentially identical to those traditionally used for evaluating shear resistance. This procedure can be seriously unconservative for large members not containing transverse reinforcement.
2. **General Procedure: Modified Compression Field Theory (MCFT)**. (Reference: LRFD Article 5.7.3.4.2 and Appendix B5). This is the recommended procedure to determine the shear resistance of concrete members.

14.3.3. Strut-and-Tie Model

Reference: LRFD Article 5.8.2.

Use the strut-and-tie model to determine internal force effects in disturbed regions, regions near reactions, applied loads, or changes in cross section, where the sectional models are not appropriate. Further, it is only applicable to the Strength and Extreme-Event limit states because significant cracking must be present for the model to be valid.

Members, when loaded, indicate the presence of definite stress fields that can individually be represented by tensile or compressive resultant forces as their vector sums. The "load paths" taken by these resultants form a truss-like pattern that is optimum for the given loading and in which the resultants are in reasonable equilibrium, especially after cracking. The objective is to conceive this optimum pattern (truss) in developing the strut-and-tie model. The closer the assumption is to this optimum pattern (truss), the more efficient the use of materials. For relatively poorly conceived strut-and-tie models, the materials will be used less efficiently, yet the structure will be safe. The compressive concrete paths are the struts, and the reinforcing steel groups are the ties. The model does not involve shear or moment because the stresses are modeled as axial loads alone.

The application of the strut-and-tie model is discussed in C5.8.2.1 of *LRFD Specifications.*

The strut-and-tie model has significant application to bridge components such as pier caps, girder ends, post-tensioning anchorage zones, etc. For a thorough presentation of the model, refer to:

- NCHRP 20-7, Task 217 Verification and Implementation of Strut-and-Tie Model in LRFD Bridge Design Specifications, November 2007;
- D. Mitchell, M. Collins, S. Bhidé and B. Rabbat, "*AASHTO LRFD Strut-and-Tie Model Design Examples,*" EB231, Portland Cement Association (PCA);
- Chapter 8 of the PCI Precast Prestressed Concrete Bridge Design Manual; and
- J. Schlaich, et al, "*Towards a Consistent Design of Structural Concrete,*" PCI Journal, Vol. 32, No. 3, 1987.

The *LRFD Specifications* provide for an adequate design; even if the strut-and-tie model is not used for actual proportioning, the model provides a fast check to ensure the adequacy of the design, especially for the appropriate anchorage of the steel.

Concrete cracking is associated with at least partial debonding of reinforcing steel bars; therefore, do not consider the bonding capacity of cracked concrete to be completely reliable. The *LRFD Specifications* generally require that reinforcing steel should not be anchored in cracked zones of concrete. Improperly anchored reinforcing steel (i.e., bars that are not fully developed) is an area that is commonly overlooked. Consider the use of headed reinforcing bars in cracked regions and where the available development length is inadequate to develop the required reinforcement strength.

14.3.4. Torsion

Torsion is not normally a major consideration in most concrete highway bridges. Where torsion effects are present, design the member in accordance with LRFD Articles 5.7.2 and 5.7.3.6. Situations that may require a torsion design include:

- cantilever brackets connected perpendicular to a concrete girder, especially if a diaphragm is not located opposite the bracket;

- concrete diaphragms used to make precast girders continuous for live load where the girders are spaced differently in adjacent spans;
- abutment caps, if they are unsymmetrically loaded; and
- horizontally curved members.

14.4. Prestressed Concrete Girders

14.4.1. General

The generic word "prestressing" relates to a method of construction in which a steel element is tensioned and anchored to the concrete. Upon release of the tensioning force, the concrete will largely be in residual compression and the steel in residual tension.

There are three methods of applying the prestressing force, as discussed below. Only two of these methods, pretensioning and post-tensioning, are acceptable, and a combination of these two methods is acceptable if approved by the Chief Bridge Engineer.

Pretensioning: In the pretensioning method, tensioning of the steel strands is complete before placing the concrete. When the concrete surrounding the steel strands attains a specified minimum strength, the strands are released thereby transmitting the prestressing force to the concrete by bond-and-wedge action at the girder ends. The initial prestress is immediately reduced due to the elastic shortening of the concrete. Further losses will occur over time due to shrinkage and creep of concrete and relaxation of prestressing steel.

The generic word "prestress" is often used to mean "pretensioning" as opposed to "post-tensioning."

Post-Tensioning: In the post-tensioning method, tensioning of the steel is accomplished after the concrete has attained a specified minimum strength. The tendons, usually comprised of numerous strands, are loaded into ducts cast into the concrete. After stressing the tendons to the specified prestressing level, they are anchored to the concrete and the jacks are released.

Several post-tensioning systems and anchorages are used in the United States; the best information may be directly obtained from the manufacturers.

Post-tensioned concrete is also subject to losses from shrinkage and creep, although at a reduced magnitude because a significant portion of shrinkage usually occurs by the time of stressing, and the rate of creep decreases with the age at which the prestress is applied. After anchoring the tendons, the ducts are pressure filled with grout, which protects the tendons against corrosion and provides composite action by bonding the strand and the girder. Post-tensioning can be applied in phases to further increase the load-carrying capacity and better match the phased dead loads being applied to the girder.

Partial Prestressing: In this hybrid design, both mild reinforcement and prestressing are present in the tension zone of a girder.

The idea of partial prestressing, at least to some extent, originated from a number of research projects that indicated fatigue problems in prestressed girders. Fatigue is a function of the stress range in the strands, which may be reduced by placing mild steel parallel to the strands in the cracked tensile zone to share live-load induced stresses. In these projects, based on a traditional model, however, the fatigue load was seriously overestimated.

The fatigue load provided by the *LRFD Specifications* is a single design vehicle with reduced weight that is not likely to cause fatigue problems unless the girder is grossly under-reinforced.

Do not use partial prestressing. Partially prestressed designs usually result in more tension in the girder at Service loads and analytical tools are not readily available to accurately predict stress-strain levels of different steels in the cross section.

14.4.2. Design Criteria for Pretensioned Concrete

This discussion applies to pretensioned concrete members.

Concrete Stress Limits

Reference: LRFD Article 5.9.42.3.

Tensile stress limits for fully prestressed concrete members must conform to the requirements for "Other Than Segmentally Constructed Bridges" in LRFD Article 5.9.42.3., except that the bridge engineer must limit the tensile stress at the Service limit state, after losses, to zero tension.

Use gross-section properties in conjunction with the Service III load combination or transformed-section properties with the Service I load combination. This requirement applies within the transfer length of the prestressing strands in addition to beyond the transfer length.

Concrete Strength at Release

Reference: LRFD Article 5.9.42.3.1.

Calculate the minimum concrete compressive strength at release (f'_{ci}) for each prestressed girder, and show it on the plans. Concrete compressive strengths at release of between 5.0 ksi and 8.0 ksi are typical. For specified concrete release strengths less than 7 ksi, round up the value shown on the plans to the next increment of 0.25 ksi. For release strengths greater than 7 ksi, round up to the next increment of 0.1 ksi.

Prestressing-Strand Stress Limits
Reference: LRFD Article 5.9.2.2.

Limit prestressing-strand stress immediately prior to transfer to 70 percent of f_{pu}.

Loss of Prestress
Reference: LRFD Article 5.9.3.

The loss of prestress is the difference between the initial stress in the strands and the effective prestress in the member. This includes both instantaneous and time-dependent losses.

Only use low relaxation strand. Estimate the time-dependent losses for decked bulb-tee girders with the following equation:

$$\Delta f_{pLT} = 33\left[1 - 0.15\left(\frac{f'_c - 6}{6}\right)\right] - 2$$

Maximum Stirrup Spacing
Do not exceed a stirrup spacing of 18 inches.

Haunch Thickness for Design
For girders supporting a cast-in-place concrete slab, consider the haunch to have maximum thickness for dead-load calculations and to be non-existent for girder-resistance calculations.

14.4.3. Precast Girder Sections

Reference: LRFD Article 5.9

This section addresses the general design theory and procedure for precast, prestressed (pretensioned) concrete girders. For additional design examples, consult the *PCI Bridge Design Manual*, Chapter 9.

Standard Girder Section
There is currently only one precast girder manufacturer in Alaska. This fabricator can produce a limited number of girder styles and sizes. Contact girder manufacturers when a desired girder shape is not readily available within the state.

1. **Precast Decked Bulb-Tee Sections.** Figure 14-3 shows the standard Alaska style precast decked bulb-tee section. The figure shows a 66-inch deep section. Fabricators create other sections by varying the member depth from 42 inches through 66 inches in 12-inch increments. Thirty six-inch deep members may also be available, depending on the number of girders required. Thirty-five-inch deep members may be available but are rarely used. Decked bulb-tee girder widths range from 49 inches up to 8½ feet but are typically between 5½ feet and 7½ feet. Table 14-4 presents the required deck reinforcing. Figure 14-4 presents the location and shape of typical bulb-tee reinforcing steel.

2. **Precast Bulb-Tee Sections.** Precast bulb-tees are available without the monolithic deck. These sections, intended to support a separate reinforced concrete deck, are identical to the precast decked bulb-tee sections, but with thinner 49-inch wide top flanges and no monolithic deck.

Girders with Cast-in-Place Decks
Design bridges with cast-in-place decks to be continuous for live load and superimposed dead loads by using a cast-in-place closure diaphragm at piers whenever possible. The design of the girders for continuous structures is similar to the design for simple spans except that, in the area of negative moments, treat the member as an ordinary reinforced concrete section. Assume that the members are fully continuous with a constant moment of inertia when determining both the positive and negative moments due to loads applied after continuity is established.

Loading Conditions
Consider the following five loading conditions in the design of a precast, prestressed girder:

1. The first loading condition is when tensioned strands are in the bed prior to placement of the concrete. Seating losses, relaxation of the strand, and temperature changes affect the stress in the strand prior to placement of the concrete. The fabricator must consider these factors during the fabrication of the girder and must adjust the initial strand tension to ensure that the tension prior to release meets the design requirements for the project. The prestressing shop drawings should present a discussion on the fabricator's proposed methods to compensate for seating losses, relaxation, and temperature changes.

2. The second loading condition is when the strands are released and the force is transferred to the concrete. After release, the girder will camber up and be supported at the girder ends only. Therefore, the region near the end of the member is not subject to bending stresses due to the dead load of the girder and may develop

Table 14-4
Standard Decked Bulb-Tee Top Flange
Required Reinforcing Steel

	Bar	Girder Width					
		≤ 6.0′	**6.0′ - 6.5′**	**6.5′ - 7.0′**	**7.0′ - 7.5′**	**7.5′ - 8.0′**	**8.0′ - 8.5′**
Interior	G401 Spaced at	8″	7″	6.5″	6″	5.75″	5.5″
Interior	G402 No. of Bars	8	10	10	10	12	14
Interior	G501 Spaced at	8″	7″	6.5″	6″	5.75″	5.5″
Interior	G502 No. of Bars	8	10	10	10	12	14
Exterior	G401X Spaced at	7″	6″	5.5″	5″	4.5″	4″
Exterior	G402X No. of bars	8	10	12	14	16	16
Exterior	G501X Spaced at	7″	6″	5.5″	5″	4.5″	4″
Exterior	G502X No. of bars	8	10	12	14	16	16

tensile stresses in the top of the girder large enough to crack the concrete. The critical sections for computing the critical temporary stresses in the top of the girder should be near the end and at all debonding points. As an option, if the bridge engineer chooses to consider the transfer length of the strands at the end of the girder and at the debonding points, then assume that the stress in the strands is zero at the end of the girder or debonding point and vary linearly to the full transfer of force to the concrete at the end of the strand transfer length.

There are several methods to relieve excessive tensile stresses near the ends of the girder:

a. harping some of the strands to reduce the strand eccentricity at the end of the girder

b. debonding, where the strands remain straight but wrapped in plastic over a predetermined distance to prevent the transfer of prestress to the concrete through bonding

c. adding additional strands in the top of the girder that are bonded at the ends but are debonded in the center portion of the girder. These strands are typically detensioned after the girder is erected.

Use the level of effective prestress immediately after release of the strands, which includes the effects of elastic shortening and the initial strand relaxation loss, to compute the concrete stresses at this stage.

3. The third loading condition may occur a few weeks to a few months after strand release when the girder is transported.

4. The fourth loading condition may occur several weeks to several months after strand release when the girder is erected and a composite deck may be cast. Camber growth and prestress losses are design factors at this stage. If a cast-in-place composite deck is placed, field adjustments to the haunch thickness are usually needed to provide the proper vertical grade on the top of deck and to keep the deck thickness uniform. The bridge engineer needs reliable estimates of deflection and camber to prevent excessive haunch thickness or to avoid significant encroachment of the top of girder into the bottom of the concrete deck. Stresses at this stage are usually not critical.

   See Section 8.7 of the *PCI Bridge Design Manual* for determining the girder camber at erection.

5. The fifth loading condition is after an extended period of time during which all prestress losses have occurred and loads are at their maximum. This is often referred to as the "maximum service load, minimum prestress" stage. The tensile stress in the bottom fibers of the girder at mid-span generally controls the design. DOT&PF policy is to limit tensile stress in the

bottom fibers of the girder to 0 ksi under the Service limit state loading.

Flexural Resistance

The design of prestressed concrete members in flexure normally begins with the determination of the required prestressing level to satisfy service conditions. Consider all load stages that may be critical during the life of the structure from the time prestressing is first applied. Follow this by a strength check of the entire member under the influence of factored loads. The strength check seldom requires additional strands or other design changes.

Intermediate Diaphragms

Reference: LRFD Article 5.12.4.

Provide cast-in-place (CIP) concrete intermediate diaphragms at the point of maximum moment for all new precast concrete girder bridges. Design the girders to support the dead load of these diaphragms. Provide openings for both planned and future utilities. At a minimum, provide an 8-inch diameter opening in each exterior girder bay for future utilities.

Figure 14-5 presents the preferred (i.e., standard) details for the CIP intermediate concrete diaphragms. Present these details on the "Typical Section" sheet or on the "Girder Details" sheet in the contract documents. Show the openings in the interior girder webs and the inserts in the exterior girder webs on the "Girder Details" sheet.

For continuous precast, prestressed girder spans, the closure diaphragms at the piers must be cast monolithically or integrally with the deck slab.

Shear Keys and Shear Connectors

Provide shear keys and shear connectors consistent with the details shown in Figure 14-6. Locate the shear connectors at 4-foot spacing along the interior top flanges.

Responsibilities

1. **Bridge Engineer.** The bridge engineer is responsible for ensuring that the proposed design will work. Select a cross section with a center of gravity (force and location), and provide a strand/tendon size and pattern to achieve the required allowable Service limit state stresses and factored flexural resistance.

   The contract documents will specify the exact value with respect to the compressive strength that the contractor must reach at release (f'_{ci}) and at 28-days (f'_c). See Section 14.4.2.

   The bridge engineer is also responsible for a preliminary investigation of shipping and handling issues where larger or long precast girders are used or where unusual site access conditions are encountered. Contact girder fabricators if shipping and handling issues appear to be unusual.

2. **Contractor.** In general, the Contractor is responsible for implementing the prestressed concrete design according to the bridge engineer's plans and specifications. The Contractor will provide shop drawings and all necessary calculations (See Chapter 25 for shop drawing checklists).

   In addition, for precast girders, the Contractor is responsible for investigating stresses in the components during proposed handling, transportation, and erection. The Contractor may propose changes to the cross sectional shape of the girder. In these cases, the Contractor must redesign the girder to meet all requirements of the project. A registered civil/structural engineer licensed in Alaska must submit design calculations and drawings for approval.

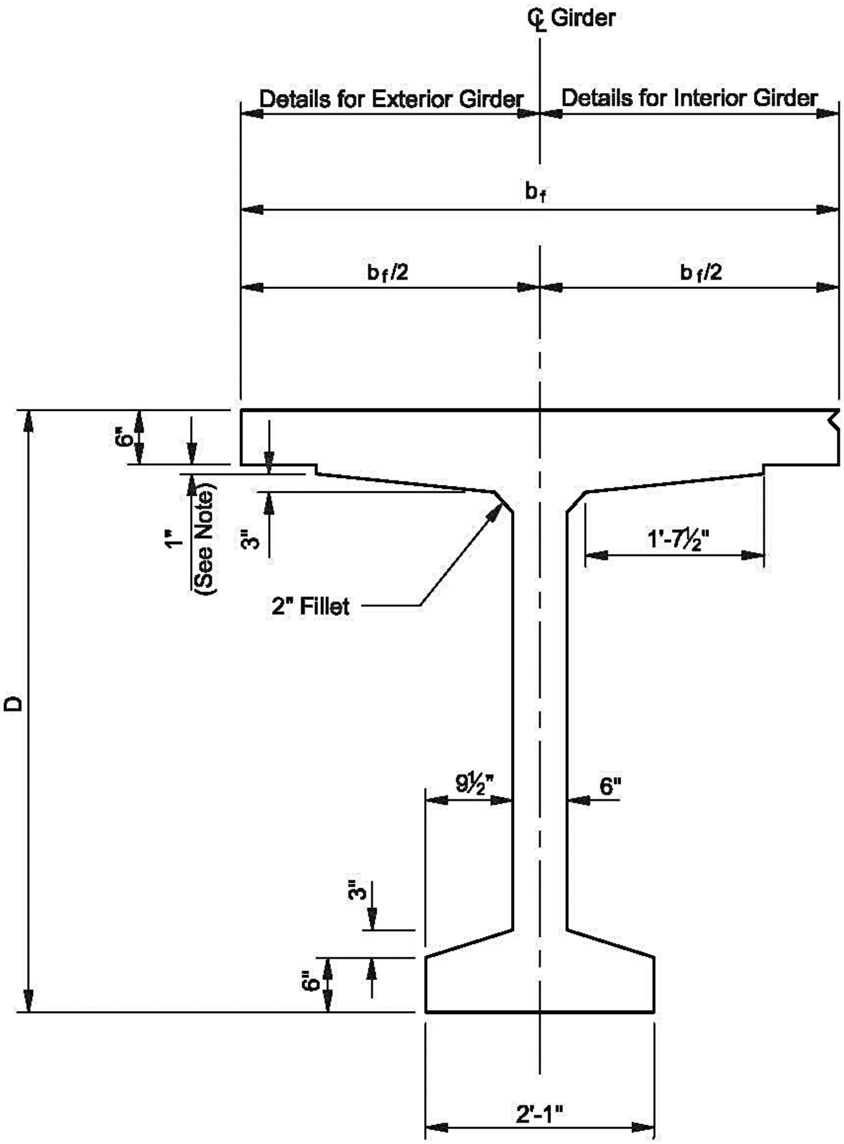
14.4.4. Cast-in-Place, Post-Tensioned Box Girders

Reference: LRFD Articles 5.12.5.

Cast-in-place, post-tensioned box girders may be used for longer span (160 feet to 500 feet) application, if they prove economical. For design requirements, see the appropriate articles of the *LRFD Specifications* and the California Department of Transportation Amendments to the AASHTO *LRFD Bridge Design Specifications*.

14.4.5. Documentation for Prestressed Concrete Girder Design Calculations

Figure 14-7 presents selected design calculations for the superstructure portion of a precast decked bulb-tee bridge. This example also provides suggested style and format for documenting design calculations.



Note: This dimension is the distinguishing feature of the Alaska style section.

Figure 14-3
Standard Alaska-Style Precast Decked Bulb-Tee Section

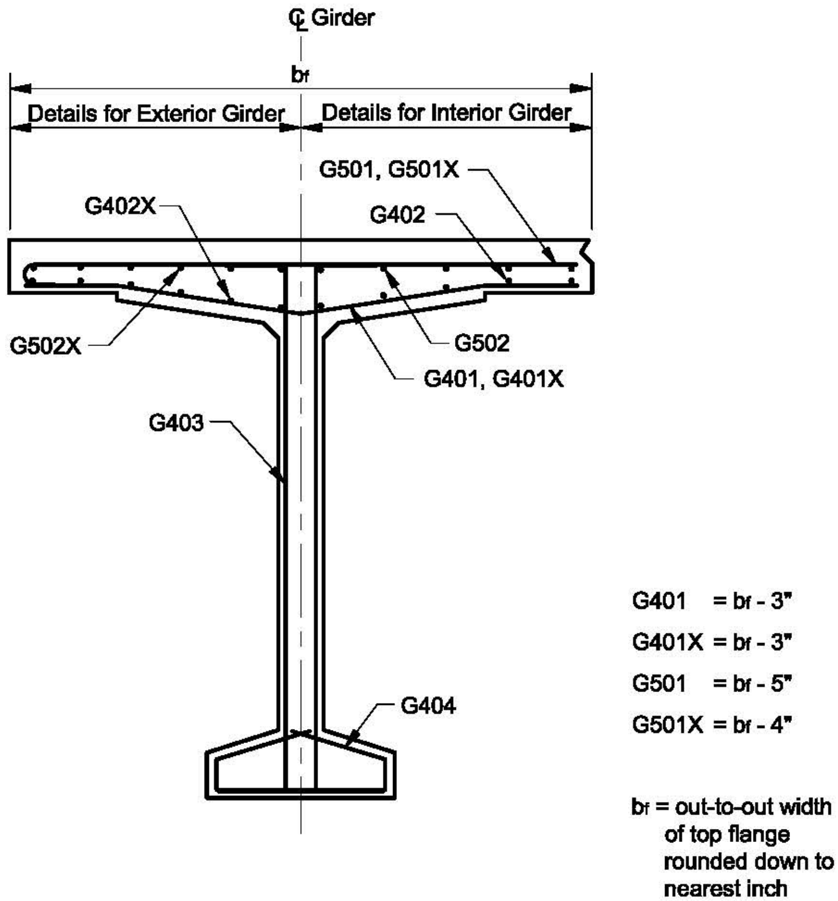
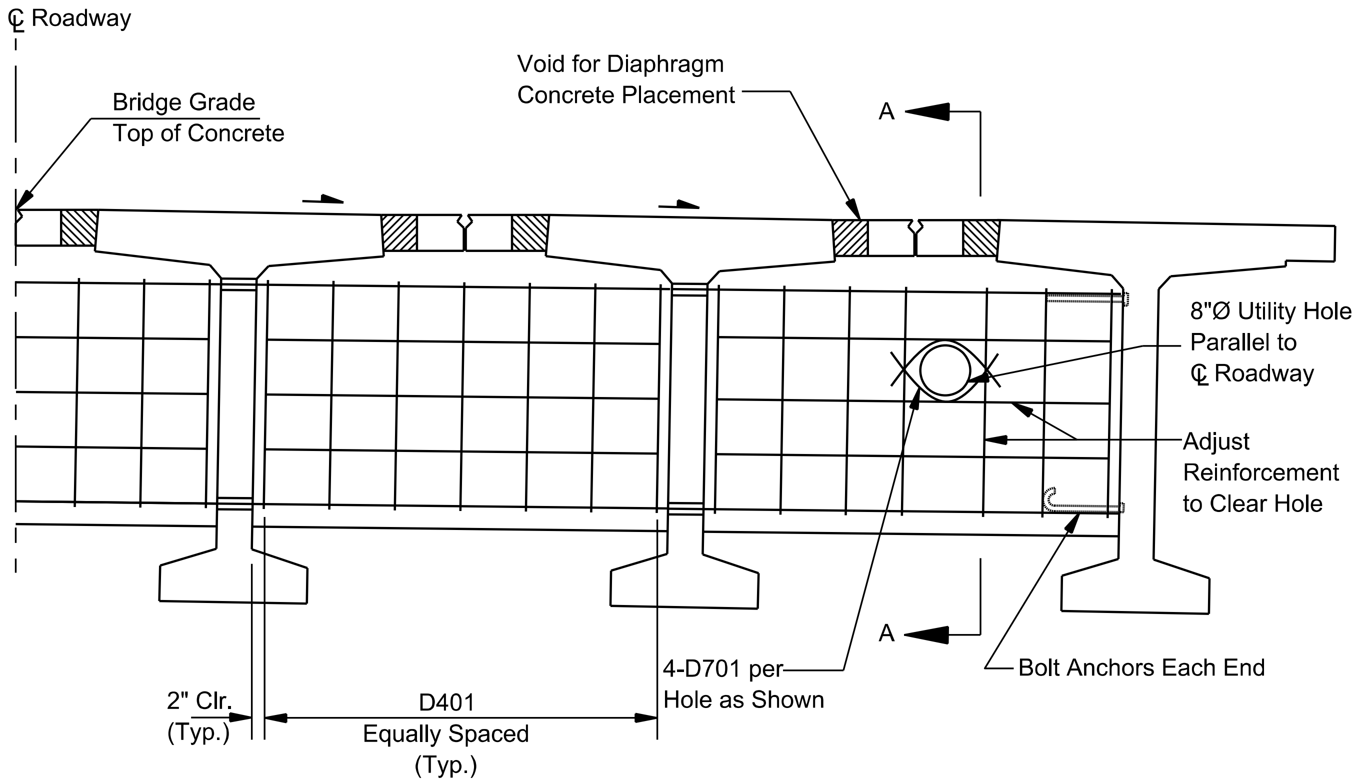
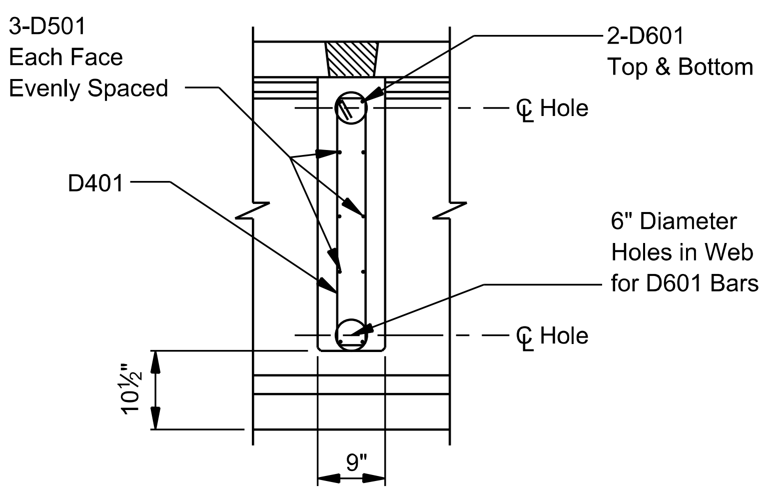


Figure 14-4
Precast Decked Bulb-Tee Typical Reinforcing Steel

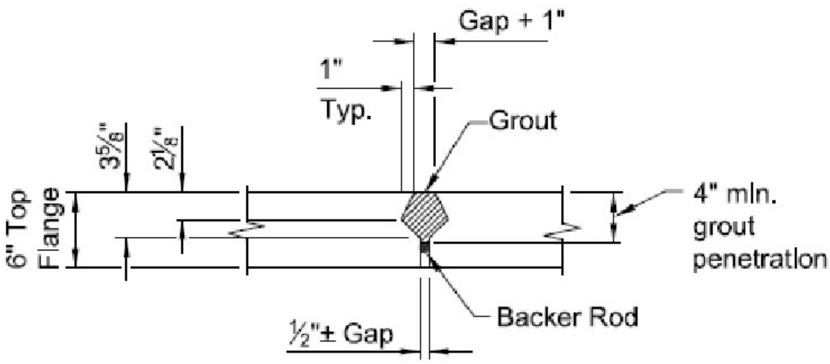


Typical Section

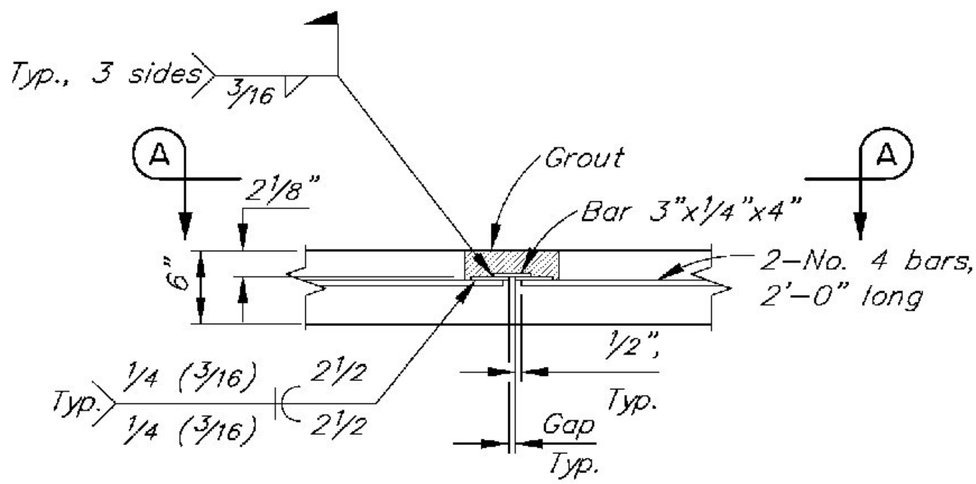


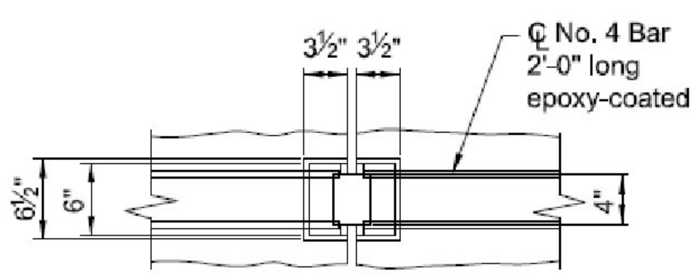
Section A-A

Figure 14-5
Intermediate Concrete Diaphragm



SHEAR KEY DETAIL





VIEW A-A

Figure 14-6
Shear Key and Shear Connector Details

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

LIVE LOADS

- For a simple span bridge with a 140.86 bearing-to-bearing length, the HL93 live load moment is:

$$M_{LL+IM} = M_{truck} \times IM + M_{LANE}$$

AASHTO 3.6.1.2

Where:

$M_{truck} = 2258$ k-ft
IM = impact allowance
$= 1.33$

$$M_{LANE} = \frac{(0.64)(140.86)^2}{8} = 1587$$

AASHTO T3.6.2.1-1

So:

$$M_{LL+IM} = (2258)(1.33) + 1587 \text{ k-ft}$$
$$\therefore M_{LL+IM} = 4590 \text{ k-ft}$$ ←

COMMENTS: Same as computer ✓ OK Near midspan

- The live load distribution factors calculated by the computer program are:

$DF_V = 0.633$
$DF_M = 0.515$

Sheet 29 of ## Sheets

Figure 14-7
Sample Calculations
(Page 1 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______

Bridge No. 2165

Calc. by EEM Date 2/10/6

Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

COMMENTS

- Verify that the moment distribution factor is reasonable with the following approximation:

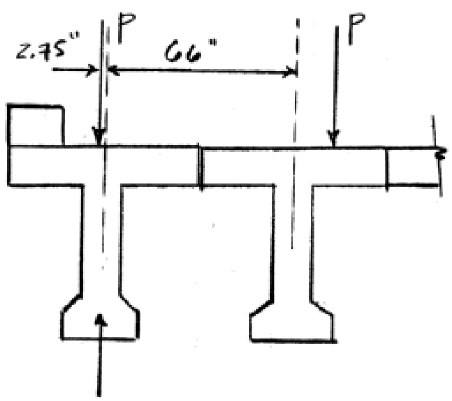
$$DF_M \cong \frac{1}{2} \times \frac{S}{5.5} \cong \frac{5.50\ ft}{11}$$

Approximation based on "old" S-over method

$\therefore DF_M \cong 0.500$ vs. 0.515 ✓ CE

3%

- and for the shear DF in the exterior girder,



AASHTO 4.6.2.2.1

$$R = \frac{68.75}{66} = 1.042\ P$$

$$DF_V = \frac{R \times m}{2P} = \frac{(1.042\ P)(1.2)}{(2)\ (P)}$$

$\therefore DF_V = 0.625$ vs. 0.633 ✓ CE

1%

Sheet 30 of ## Sheets

Figure 14-7
Sample Calculations
(Page 2 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
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Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

PRESTRESS LOSSES

- Verify that the prestress losses calculated by the computer program are reasonable.

$$\Delta f_{PT} = \Delta f_{PES} + \Delta f_{PLT}$$

AASHTO 5.9.5.1

Where:

Δf_{PT} = Total short and long term loss

Δf_{PES} = Elastic shortening loss

$= \frac{E_P}{E_{c_i}} f_{cGP}$

AASHTO 5.9.5.2.3.a
See Sheet 24

E_P = 28500 ksi

E_{ci} = 5185 ksi

f_{cGP} = Concrete stress at C.G. of prestressing and DL self weight

$= \frac{-P_i}{A} - \frac{P_i \times e}{S_{cG}} + \frac{M_{DG1}}{S_{cG}}$

P_i = Initial P/S after Δf_{PES}

$= [(0.7)(270) - \Delta f_{PES}](0.153)(N_s)$

N_s = No. of strands = 64

See Sheet 25

A = 1013 in^2

S_{cG} = 15577 in^3

M_{PCI} = 2728 k-ft = 32736 k-in

e = 36.89 in

See Sheet 28

COMMENTS

Sheet 31 of ## Sheets

Figure 14-7
Sample Calculations
(Page 3 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

Try, $\Delta f_{pES} = 19.1$ ksi

So, $P_i = (189 - 19.1)(0.153)(64) = 1663.7$

And,

$$f_{cgp} = \underbrace{\frac{-1663.7}{1013}}_{-1.642} - \underbrace{\frac{(1663.7)(36.89)}{15577}}_{-3.940} + \underbrace{\frac{32736}{15577}}_{+2.102}$$

$f_{cgp} = 3.480$ ksi [compression]

Then,

$$\Delta f_{pES} = \frac{28500}{5185}(3.480 \text{ ksi})$$

✓OK

$\therefore \Delta f_{pES} = 19.1$ ksi -- Same as trial ←

COMMENTS

ALSO THE SAME AS THE COMPUTER ∴GOOD

Sheet 32 of ## Sheets

Figure 14-7
Sample Calculations
(Page 4 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
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PUBLIC FACILITIES

Computations

For: ANTLER SLOUGH BRIDGE

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

	COMMENTS
• and for the long-term P/S losses using the approximate method:	AASHTO T.5.9.5.3-1
$$\Delta f_{PLT} = 33.0\left[1 - \frac{0.15\,(f'_c - 6)}{6}\right] + 6PPR - 8^{*}$$	* FOR LOW-RELAXATION STRAND
Where:	
$f'_c = 7.5$ ksi	
$PPR = 1.0$ for P/S girders	
So:	
$$\Delta f_{PLT} = 33\left[1 - \frac{(0.15)(1.5)}{6}\right] - 2$$	
$\therefore \Delta f_{PLT} = 29.8$ ksi ←	SAME AS COMPUTER VALUE
• The total P/S losses are:	
$\Delta f_{PT} = 19.1 + 29.8$	
$\therefore \Delta f_{PT} = 48.9$ ksi ←	SAME AS PROGRAM

Sheet 33 of ## Sheets

Figure 14-7
Sample Calculations
(Page 5 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ________
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by _____ Date _____

For: ANTLER SLOUGH BRIDGE

COMMENTS

- The prestress forces at various times:

Jacking

$f_j = (0.7)(270) = 189 \text{ ksi}$

$P_j = (64)(0.153)(189) = 1851 \text{ k}$ ←

Release

$f_i = 189 - 19.1 = 169.9 \text{ ksi}$

$P_i = (64)(0.153)(169.9) = 1664 \text{ k}$ ←

Final

$f_f = 169.9 - 29.8 = 140.1 \text{ ksi}$

$P_f = (64)(0.153)(140.1) = 1372 \text{ k}$ ←

Sheet 34 of ## Sheets

Figure 14-7
Sample Calculations
(Page 6 of 6)

14.5. References

Goodnight, J.C., Kowalsky, M.J., and Nau, J.M., 2015. *The Effects of Load History and Design Variables on Performance Limit States of Circular Bridge Columns*, volumes 1 and 2, Report No. 4000(72), Alaska DOT&PF, Juneau, AK, 278 and 748 pp.

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