Chapter 14

CONCRETE STRUCTURES

NDOT STRUCTURES MANUAL

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Chapter 14
CONCRETE STRUCTURES

Section 5 of the *LRFD Bridge Design Specifications* presents unified design requirements for concrete, both reinforced and prestressed, in all structural elements. The American Concrete Institute (ACI) similarly uses unified provisions in ACI 318. This Chapter presents NDOT supplementary information specifically on the properties of concrete and reinforcing steel and the design of structural concrete members.

14.1 STRUCTURAL CONCRETE DESIGN

14.1.1 Member Design Models

Reference: LRFD Articles 5.6.3, 5.8.1, 5.8.3 and 5.13.2

Where it is reasonable to assume that a planar section remains planar after loading, the *LRFD Specifications* allows two approaches to the design for concrete members the strut-and-tie model and the traditional sectional design model. Their basic application is as follows:

1. Sectional Design Model. The sectional design model is appropriate for the design of typical bridge girders, slabs and other regions of components where the assumptions of traditional girder theory are valid. This 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 5.8.3 discusses the sectional design model. Subarticles 1 and 2 describe the applicable geometry required to use this technique to design for shear.

2. Strut-and-Tie Model. The strut-and-tie model should be used in regions near discontinuities (e.g., abrupt changes in cross section, openings, coped (dapped) ends, deep girders, corbels). See LRFD Articles 5.6.3 and 5.13.2.

The following Sections discuss each of these member design approaches.

14.1.2 Sectional Design Model

Reference: LRFD Article 5.8.3

14.1.2.1 Flexural Resistance

Reference: LRFD Article 5.7

The flexural resistance of a girder section is typically obtained using the rectangular stress distribution of LRFD Article 5.7.2.2. In lieu of using this simplified, yet accurate approach, a strain compatibility approach may be used as outlined in LRFD Article 5.7.3.2.5. The general equation for structural concrete flexural resistance of LRFD Article 5.7.3.2.1 is based upon the rectangular stress block.

14.1.2.2 Limits for Flexural Steel Reinforcement

14.1.2.2.1 *Maximum*

Reference: LRFD Articles 5.7.3.3.1 and 5.5.4.2.1

The current LRFD provisions eliminate the traditional maximum limit of reinforcement. Instead, a phi-factor varying linearly between the traditional values for flexure and compression members represented by LRFD Equations 5.5.4.2.1-1 or 5.5.4.2.1-2 is applied to differentiate between tension- and compression-controlled sections.

14.1.2.2.2 *Minimum*

Reference: LRFD Articles 5.7.3.3.2 and 5.4.2.6

The minimum flexural reinforcement of a component should provide flexural strength at least equal to the lesser of:

- 1.2 times the cracking moment of the concrete section, defined by LRFD Equation 5.7.3.3.2-1 and assuming that cracking occurs at the Modulus of Rupture, taken as $0.37\sqrt{f'_c}$ for normal-weight concrete; or

- 1.33 times the factored moment required by the governing load combination.

14.1.2.3 Distribution of Reinforcement

Reference: LRFD Article 5.7.3.4

In addition to the provisions of LRFD Article 5.7.3.4, the following will apply:

1. Negative Moments. For the distribution of negative moment tensile reinforcement continuous over a support, the effective tension flange width should be computed separately on each side of the support in accordance with LRFD Article 5.7.3.4. The larger of the two effective flange widths should be used for the uniform distribution of the reinforcement into both spans.

2. Girders. Within the negative moment regions of continuous cast-in-place structures, the top side face bar on each face of the girder web shall be #8 bar.

3. Integral Pier Caps. For integral pier caps, reinforcement shall be placed approximately 3 in below the construction joint between the deck and cap, or lower if necessary to clear prestressing ducts. This reinforcement shall be designed by taking M_u as 1.3 times the dead load negative moment of that portion of the cap and superstructure located beneath the construction joint and within 10 ft of each side face of the cap. Service load checks and shear design are not required for this condition. This reinforcement may be included in computing the flexural capacity of the cap only if a stress and strain compatibility analysis is made to determine the stress in the bars.

14.1.2.4 Crack Control Reinforcement

Reference: LRFD Article 5.7.3.4

Reinforcing bars in all reinforced concrete members in tension shall be distributed to control cracking in accordance with LRFD Article 5.7.3.4. When designing for crack control, the following values shall be used, unless a more severe condition is warranted:

- γ_e = 0.75 (Class 2 exposure condition) 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) for all other components.

Several smaller reinforcing bars at moderate spacing are more effective in controlling cracking than fewer larger bars.

14.1.2.5 Shear Resistance

Reference: LRFD Article 5.8.3

14.1.2.5.1 *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* presents two alternative sectional shear design models for estimating the shear resistance of concrete members.

<u>General Procedure: Modified Compression Field Theory (MCFT)</u>

Reference: LRFD Article 5.8.3.4.2

The nominal shear resistance is taken as the lesser of:

$$V_n = V_c + V_s + V_p \qquad \text{(LRFD Eq. 5.8.3.3-1)}$$
$$V_n = 0.25 f'_c b_v d_v + V_p \qquad \text{(LRFD Eq. 5.8.3.3-2)}$$

For non-prestressed sections, $V_p = 0$.

LRFD Equation 5.8.3.3-2 represents an upper limit of V_n to ensure that the concrete in the web will not crush prior to yielding of the transverse reinforcement.

The nominal shear resistance provided by tension in the concrete is computed by:

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

The contribution of the web reinforcement is given by:

$$V_s = \frac{A_v f_y d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s} \qquad \text{(LRFD Eq. 5.8.3.3-4)}$$

where the angles, θ and α, represent the inclination of the diagonal compressive forces measured from the longitudinal axis and the angle of the web reinforcement relative to the longitudinal axis, respectively.

For the usual case where the web shear reinforcement is vertical (α = 90°), V_s simplifies to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

Both θ and β are functions of the longitudinal steel strain (ε_x) which, in turn, is a function of θ. Therefore, the design process is an iterative one. LRFD Article 5.8.3.4.2 provides the detailed methodology and the design tables. For sections containing at least the minimum amount of transverse reinforcement specified in LRFD Article 5.8.2.5, the values of β and θ should be taken from LRFD Table 5.8.3.4.2-1. For sections that do not meet the minimum transverse reinforcement requirements, LRFD Table 5.8.3.4.2-2 should be used to determine β and θ.

Sections meeting the requirements of LRFD Article 5.8.3.4.1 may be designed using a value of 2.0 for β and a value of 45° for θ. However, these traditional values of θ and β have proven seriously unconservative for large members not containing transverse reinforcement (footings, for example).

Transverse shear reinforcement shall be provided when:

$$V_u > 0.5\,\phi\left(V_c + V_p\right) \qquad \text{(LRFD Eq. 5.8.2.4-1)}$$

Where transverse reinforcement is required, the area of steel shall not be less than:

$$A_v = 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(LRFD Eq. 5.8.2.5-1)}$$

For the usual case where the reaction introduces compression into the end of the member, the critical section for shear is taken as d_v, measured from the face of the support (see LRFD Article 5.8.3.2).

The sectional model requires a check of the adequacy of the longitudinal reinforcement in LRFD Article 5.8.3.5. This requirement acknowledges that shear causes tension in the longitudinal reinforcement. All steel on the flexural tension side of the member, prestressed and non-prestressed, may be used to satisfy this requirement.

Simplified Procedure

Reference: LRFD Article 5.8.3.4.3

The simplified procedure is similar to the traditional approach in the *Standard Specifications*. In this procedure, the lesser of two components, V_{ci} and V_{cw}, is used to quantify the concrete contribution to shear resistance. Although this procedure is not iterative, it can be more conservative than the MCFT approach.

14.1.2.5.2 *Shear Friction*

Reference: LRFD Article 5.8.4

The steel required to comply with the provisions of LRFD Article 5.8.4 shall be considered additive to the steel required from other analyses, except as provided for in LRFD Article 5.10.11.

14.1.3 Strut-and-Tie Model

Reference: LRFD Article 5.6.3

The strut-and-tie model is used 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.

This method of modeling concrete components originated around 1900, but it has only recently been incorporated into the AASHTO bridge design code. Members, when loaded, indicate the presence of definite stress fields that can individually be represented by tensile or compressive resultant forces as their vectorial sums. It has been observed that the "load paths" taken by these resultants form a truss-like pattern that is optimum for the given loading and that the resultants are in reasonable equilibrium, especially after cracking. The designer's objective is to conceive this optimum pattern (truss) in developing the strut-and-tie model. The closer the designer's 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 encompasses several simple steps:

1. The truss model must be envisioned that carries the applied loads to the reactions and, subsequently, the truss geometry established.

2. The struts are proportioned according to the provisions of LRFD Article 5.6.3.3, and the ties according to LRFD Article 5.6.3.4.

3. The nodal regions connecting the truss members are proportioned according to the provisions of LRFD Article 5.6.3.5, wherein concrete compression stresses are limited.

4. Finally, crack control reinforcement is provided according to LRFD Article 5.6.3.6 to control the significant cracking necessary to facilitate the strut-and-tie model.

The strut-and-tie model has significant application to bridge components such as pier caps, girder ends, post-tensioning anchorage zones, etc. A thorough presentation of the model can be found in:

- 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* provides adequately for 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.

Cracking is associated with at least partial debonding and, thus, the bonding capacity of cracked concrete cannot be considered completely reliable. The *LRFD Specifications* generally requires that reinforcing steel should not be anchored in cracked zones of concrete. Improperly anchored reinforcing steel is an area that is commonly overlooked.

14.1.4 Fatigue

Reference: LRFD Articles 3.4.1, 3.6.1.4 and 5.5.3

The fatigue limit state is not normally a critical issue for concrete structures. Fatigue need not be considered for decks nor where the permanent stress f_{min} is compressive and exceeds twice the maximum tensile live load stress. Also, fatigue need not be considered for strands in fully prestressed concrete members.

Assuming r/h = 0.3, LRFD Equation 5.5.3.2-1 for mild reinforcement may be rearranged for easier interpretation:

$$f_f + 0.33\, f_{min} \leq 24 \text{ ksi}$$

14.1.5 Torsion

Reference: LRFD Article 5.8

Torsion is not normally a major consideration in most highway bridges. Where torsion effects are present, the member shall be designed in accordance with LRFD Articles 5.8.2 and 5.8.3.6. 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; and
- abutment caps, if they are unsymmetrically loaded.

14.2 MATERIALS

14.2.1 Structural Concrete

Reference: LRFD Article 5.4.2.1

Figure 14.2-A presents NDOT criteria for the minimum compressive strength of concrete in structural elements.

Structural Element		**Minimum 28-Day Compressive Strength (f'_c)**
Bridge Decks, Approach Slabs and Barrier Rails	Clark County	4 ksi
	Rest of State	4.5 ksi
CIP Concrete Slabs	Clark County	4 ksi
	Rest of State	4.5 ksi
Prestressed Concrete (Post-Tensioned)		4 ksi*
Prestressed Concrete (Precast)		5 ksi*
Piers and Columns		4 ksi
Abutments		4 ksi
Wingwalls		4 ksi
Spread Footings		4 ksi
Drilled Shafts		4 ksi
Reinforced Concrete Boxes (non-standard)		4 ksi

* *The maximum strength for post-tensioned and precast, prestressed concrete shall not exceed 6.5 ksi and 7.5 ksi, respectively. Higher strengths shall not be used without the approval of the Chief Structures Engineer and a review by the NDOT Materials Division*.

COMPRESSIVE STRENGTH OF CONCRETE

Figure 14.2-A

14.2.2 Reinforcing Steel

Reference: LRFD Article 5.4.3.1

For general application, reinforcing steel shall conform to the requirements of ASTM A615, Grade 60. For seismic applications, reinforcing steel shall conform to the requirements of ASTM A706, Grade 60. The modulus of elasticity, E_s, is equal to 29,000 ksi.

Where reinforced concrete elements are designed to resist seismic forces beyond the elastic limit of the reinforcing steel, the bridge designer shall specify A706, Grade 60 reinforcing steel. ASTM A706 reinforcing steel is manufactured with controlled material properties. These properties include a maximum yield strength and a minimum ratio between the tensile and yield strengths. In addition, ASTM A706 reinforcing steel is manufactured with a controlled chemical composition making it more weldable. All welding of this reinforcing steel should be in accordance with AWS D1.4.

If A706 reinforcing steel is specified for elements in a bridge, it should be used for the entire bridge. This eliminates the need for separate inventories and increased inspection at the job site.

Reinforcing steel with a yield strength greater than 60 ksi may be used with the approval of the Chief Structures Engineer for minor structures (e.g., culverts, sound walls). However, the design must satisfy all limit states, including serviceability (i.e., cracking). Do not exceed a strength greater than 75 ksi as the basis for design.

14.2.3 Welded Wire Reinforcement

Welded Wire Reinforcement (WWR), also referred to as welded wire fabric, is an alternative to conventional concrete reinforcing steel for approved applications; see Section 14.3.2. WWR is prefabricated in a series of parallel wires welded with cross wires to form square or rectangular grids. All wires in one direction are the same diameter, but the wires in the transverse direction are commonly different. Each wire intersection is resistance welded.

The wires used in WWR can be either smooth or deformed. WWR generally used for MSE soil reinforcement and precast reinforced concrete box culverts conforms to AASHTO M55 (ASTM A185) “Steel Welded Wire Reinforcement, Plain, for Concrete.” WWR used in structural applications, such as precast soundwall panels, conforms to AASHTO M221 (ASTM A497) “Steel Welded Wire Reinforcement, Deformed, for Concrete.”

14.2.4 Prestressing Strand

Reference: LRFD Article 5.4.4.1

Prestressing strand shall be low-relaxation, 7-wire strand with a minimum tensile strength of f_{pu} = 270 ksi and a minimum yield strength of f_{py} = 243 ksi. The minimum modulus of elasticity, E_p, is equal to 28,500 ksi.

14.2.5 Prestressing Bars

Reference: LRFD Article 5.4.4.1

Prestressing bars shall be plain or deformed bars with a minimum tensile strength of f_{pu} = 150 ksi, with a yield strength of 127.5 ksi for plain bars and 120 ksi for deformed bars. The minimum modulus of elasticity, E_p, is equal to 30,000 ksi.

14.3 REINFORCEMENT

14.3.1 Reinforcing Steel

14.3.1.1 Bar Sizes

Reinforcing steel is referred to in the bridge plans and specifications by number, and they vary in size from #3 to #18 in US Customary units. Figure 14.3-A presents the sizes and properties of the bars used by NDOT.

Bar Size Designation		Nominal Dimensions		
US Customary	Metric	Weight (lbs/ft)	Diameter (in)	Area (in^2)
#3	#10	0.376	0.375	0.11
#4	#13	0.668	0.500	0.20
#5	#16	1.043	0.625	0.31
#6	#19	1.502	0.750	0.44
#7	#22	2.044	0.875	0.60
#8	#25	2.670	1.000	0.79
#9	#29	3.400	1.128	1.00
#10	#32	4.303	1.270	1.27
#11	#36	5.313	1.410	1.56
#14	#43	7.650	1.693	2.25
#18	#57	13.600	2.257	4.00

REINFORCING STEEL SIZES

Figure 14.3-A

14.3.1.2 Concrete Cover

Reference: LRFD Article 5.12.3

Figure 14.3-B presents NDOT criteria for minimum concrete cover for various applications. These are the minimums regardless of the w/c ratio. All clearances to reinforcing steel shall be shown in the bridge plans.

14.3.1.3 Spacing of Bars

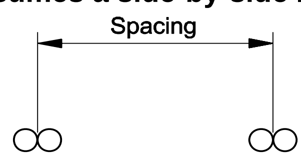
Reference: LRFD Article 5.10.3

Figure 14.3-C presents NDOT criteria for minimum spacing between reinforcement bars based on bar size and spliced vs unspliced. The accompanying sketch illustrates how to measure the spacing for spliced bars.

Structural Element or Condition		Minimum Concrete Cover
Concrete Deck Slabs	Top	2½″
	Bottom	1½″
Exposed to Deicing Salts (Barrier Rails, Approach Slabs, Top of Pier Caps, Abutment Seats)		2½″
Top of Pier Caps not Exposed to Deicing Salts		2″
Drilled Shafts (Diameter ≥ 3′)		6″
Drilled Shafts (Diameter < 3′)		4″
Stirrups and Ties		1½″
Reinforced Concrete Boxes	General	2″
	Against Ground	2½″
Formed Concrete Not Exposed to Ground		1½″
Formed Concrete Exposed to Ground		2″
Concrete Cast Against Ground		3″
Precast Members (Mild Reinforcement)		1½″

CONCRETE COVER

Figure 14.3-B

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

MINIMUM SPACING OF BARS

Figure 14.3-C

Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings. Skews will tend to complicate problems with reinforcing fit. Tolerances normally allowed for cutting, bending and locating reinforcing should be considered. Refer to ACI 315 for allowed tolerances. Some of the common areas of interference are:

- anchor bolts in abutment caps;
- between slab reinforcing and reinforcing in monolithic abutments or piers;
- vertical column bars projecting through main reinforcing in pier caps;
- the areas near expansion devices;
- embedded plates for prestressed concrete girders;
- anchor plates for steel girders;
- at anchorages for a post-tensioned system; and
- between prestressing (pretensioned or post-tensioned) steel and reinforcing steel stirrups, ties, etc.

14.3.1.4 Fabrication Lengths

Use a maximum length of 60 ft for detailing reinforcing steel.

14.3.1.5 Lateral Confinement Reinforcement

14.3.1.5.1 *Columns*

Reference: LRFD Article 5.10.11.4

All lateral column reinforcement shall be detailed for Zones 3 and 4 requirements in LRFD Article 5.10.11.4. Lateral reinforcement for compression members shall consist of either spiral reinforcement, welded hoops or a combination of lateral ties and cross ties. Ties shall only be used when it is not practical to provide spiral or hoop reinforcement. Where longitudinal bars are required outside the spiral or hoop reinforcement, they shall have lateral support provided by bars spaced and hooked as required for cross ties. The hooked bars shall extend into the core of the spiral or hoop a full development length.

14.3.1.5.2 *Drilled Shafts*

The reinforcing steel cage for drilled shafts shall extend the full length of the pile.

The length of the plastic hinge confinement reinforcement shall be determined by appropriate analysis but shall not be less than the requirements of LRFD Article 5.13.4.6.3d.

The designer should 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 maximum spacing requirements of LRFD Article 5.13.6.3d shall be maintained.

14.3.1.5.3 *Headed Reinforcement*

Headed reinforcement can be considered as an alternative to lateral reinforcing steel when conflicts make the use of tie reinforcement impractical. Headed reinforcement consisting of friction welded or internally forged heads shall conform to ASTM A970/A970M.

14.3.1.6 Corrosion Protection

Epoxy-coated bars are not used in Clark County. For projects in the remainder of the State, the following presents NDOT policies for the use of epoxy-coated reinforcement bars:

- bridge decks (both layers),
- reinforcing that extends into bridge decks and/or terminates within 12 in of the top of the deck slab,
- cap shear and primary reinforcement of caps and abutments located under deck joints,
- integral cap shear and top reinforcement,
- bridge approach slabs,
- wingwalls (if not covered by an approach slab),
- barrier rails, and
- sidewalks.

14.3.1.7 Development of Reinforcement

Reinforcement must be developed on both sides of a point of maximum stress at any section of a reinforced concrete member. This requirement is specified in terms of a development length, l_d.

14.3.1.7.1 *Development Length in Tension*

Reference: LRFD Article 5.11.2

The development of bars in tension involves calculating the basic development length, l_{db}, which is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top bar effect, type of aggregate, and the ratio of required area to provide the area of reinforcement to be developed.

The development length, l_d (including all applicable modification factors), must not be less than 12 in.

14.3.1.7.2 *Development Length in Compression*

Columns shall not be considered compression members for development length computations. When designing column bars with hooks to develop the tension, ensure that the straight length is also adequate to develop the bar in compression because hooks are not considered effective in developing bars in compression. This practice ensures that columns in bending will have adequate development in both tension and compression.

14.3.1.7.3 *Standard End Hook Development Length in Tension*

Reference: LRFD Article 5.11.2.4

Standard hooks use a 90° and 180° 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.

Refer to the figure in the commentary of LRFD Article C5.11.2.4.1 for hooked-bar details for the development of standard hooks. Use the same figure for both uncoated and coated bars, modified as appropriate by the factors noted in Section 14.3.1.7.1.

14.3.1.8 Splices

Reference: LRFD Article 5.11.5

14.3.1.8.1 *Types/Usage*

The following presents NDOT practices on the types of splices and their usage:

1. <u>Lap Splices</u>. NDOT uses conventional lap splices whenever practical. Use the Standard Minimum Lap Splice Lengths shown in Figure 14.3-D for all tension and compression lap splices unless a longer splice length is required by calculation. It is NDOT practice to use as a minimum a Class C splice for #4 through #8 bars and a Class B splice for #9 through #11 bars. Where feasible, stagger lap splices for main-member reinforcement such that no more than 50% are lapped in any one location. A minimum stagger of 2 ft between adjacent centerlines of splices is required for individual and bundled bars.

   If transverse reinforcing steel in a bridge deck is lapped near a longitudinal construction joint, the entire lap splice shall be placed on the side of the construction joint that will be poured last.

2. <u>Mechanical Splices</u>. (Reference: LRFD Articles 5.11.5.2.2, 5.11.5.3.2 and 5.11.5.5.2). A second method of splicing is by mechanical splices, which use proprietary splicing mechanisms. Mechanical splices are appropriate away from plastic hinges and where interference problems preclude the use of more conventional lap splices, and in staged construction. Even with mechanical splices, it is frequently necessary to stagger splices. The designer must check clearances. In addition, fatigue shall be considered. Mechanical splices shall develop 125% of the bar yield strength for reinforcing steel in non-yielding areas. Mechanical splices shall develop 160% of the bar yield strength for reinforcing steel in yielding areas not subject to plastic hinging.

Bar Size	Area (in^2)	Diameter (in)	Class	Uncoated (in)	Epoxy Coated (in)
#4	0.20	0.500	C	21	25
#5	0.31	0.625	C	26	31
#6	0.44	0.750	C	31	37
#7	0.60	0.875	C	39	46
#8	0.79	1.000	C	51	61
#9	1.00	1.128	B	49	59
#10	1.27	1.27	B	62	75
#11	1.56	1.41	B	77	92

Note: Lap splice lengths based on f'_c = 4 ksi, f_y = 60 ksi, non-top bars, uncoated bars spaced less than 6 in or with a clear cover of less than 3 in, epoxy coated bars spaced more than 6 bar diameters or with a clear cover of more than 3 bar diameters, and normal weight concrete.

STANDARD MINIMUM SPLICE LENGTHS FOR BARS IN TENSION AND COMPRESSION

Figure 14.3-D

3. Welded Splices. Splicing of reinforcing bars by welding, although allowed by the *LRFD Specifications*, is seldom used by NDOT and not encouraged primarily because of quality issues with field welding. However, shop-fabricated, butt-welded hoops can be used as confinement reinforcement for columns. Welding of reinforcing steel is not addressed by the AASHTO/ANSI/AWS *D1.5 Bridge Welding Code*, and the designer must reference the current *Structural Welding Code Reinforcing Steel* of AWS (D1.4).

4. Full Mechanical/Welded Splices. See LRFD Article 5.11.5.3.2.

14.3.1.8.2 *Plastic Hinge Regions*

In columns and drilled shafts, there shall be no splices in the longitudinal reinforcing or splicing of spiral reinforcing within the plastic hinge regions. These regions shall be clearly identified in the contract documents.

14.3.1.9 Bundled Bars

Reference: LRFD Articles 5.11.2.3 and 5.11.5.2.1

NDOT allows the use of two-bundled or three-bundled bars; NDOT prohibits the use of four-bundled bars.

The development length of bars within a bundle shall be taken as that of an individual bar as specified in Section 14.3.1.7, increased by 20% for a three-bar bundle.

Lap splices of bundled bars shall be based upon development lengths as specified above. Entire bundles shall not be lap spliced at the same location. Individual bars within a bundle may be lap spliced, but the splices shall not overlap. Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings.

14.3.2 <u>Welded Wire Reinforcement (WWR)</u>

14.3.2.1 Design and Detailing

Concrete cover, development length and lap length for WWR shall meet the requirements of the *LRFD Specifications*. For corrosion protection, WWR can be provided with an epoxy coating conforming to ASTM A884, Class A.

Standard practice for detailing WWR can be found in the Wire Reinforcement Institute's *Manual of Standard Practice, Structural Welded Wire Reinforcement*. This document also provides commonly available wire sizes, spacing and available mat lengths.

14.3.2.2 Application and Limitations

Welded wire reinforcement may be considered as a substitute for AASHTO M31 (ASTM A615) reinforcing steel for minor structural applications, including:

- precast box culverts,
- precast MSE wall panels,
- precast sound barrier panels,
- drainage structures and appurtenances,
- channel linings, and
- slope paving.

Approval by the Chief Structures Engineer is required for all other applications. WWR shall not be used as a substitute for ASTM A706 reinforcing steel.

The following applies to the usage of WWR:

1. WWR can be provided as a direct replacement, with equivalent cross-sectional area, for the specified reinforcing steel. This is the preferred method of substitution.

2. WWR can be provided as a proposed redesign to take advantage of the higher yield strength of WWR. Supporting calculations and drawings sealed by a registered Nevada professional civil/structural engineer shall be submitted for approval. The design must satisfy all limit states, including serviceability (e.g., cracking). Yield strengths in excess of 75.0 ksi shall not be used for design purposes.

Material certifications must also be provided; the bridge designer should consult with the Materials Division to determine appropriate testing.

14.3.3 Prestressing Strands and Tendons

14.3.3.1 Pretensioned Girders

14.3.3.1.1 *Strand Size*

Common sizes of prestressing strand used in bridge construction are ½-in and 0.6-in diameter. The preferred diameter of the prestressing strands in pretensioned girders is ½ in.

14.3.3.1.2 *Strand Spacing*

The minimum spacing of strands shall not be less than 2 in center to center.

14.3.3.1.3 *Strand Profile*

It is acceptable to use either a straight or draped strand profile for precast members. NDOT prefers "draped" strand (i.e., deviated, harped, deflected) to "debonded," because of the greater shear capacity. However, a combination of debonded and draped strands may be used when necessary to satisfy design requirements. The advantages of straight trajectories include their simplicity of fabrication and greater safety. Debonded or draped strands are used to control stresses and camber. Debonded strands are easier to fabricate because a hold-down point is not required in the stressing bed. For debonded strands, see Section 14.5.4.4.

14.3.3.1.4 *Draped Strand*

The following applies to draped strands in precast, pretensioned girders:

- At ends of girders, maintain a minimum of 4 in between the top draped strands and any straight strands that are located directly above the draped strands.
- At each hold-down point, the vertical force should be limited to a maximum of 48 kips for all draped strands and 4 kips for each individual draped strand.
- The slope of the draped strands should not exceed 9°.
- Where practical, hold-down points should be located 5 ft on each side of the centerline of the girder (10 ft apart).

14.3.3.1.5 *Strand Patterns*

The designer must fully detail the strand pattern showing the total number of strands, layout and spacing, edge clearances, which strands will be draped and/or debonded, and the layout of all mild reinforcing steel. Frequently, precast, pretensioned girders of the same size and similar length in the same bridge or within bridges on the same project may be designed with a slightly different number of strands. In this case, the designer should consider using the same number and pattern of strands (including height of draping) for these girders to facilitate fabrication.

14.3.3.1.6 *Strand Splicing*

Splicing of prestressing strand is not allowed.

14.3.3.2 Post-Tensioned Members

14.3.3.2.1 *Strand Size*

The preferred diameter of the prestressing strand used for post-tensioning is 0.6 in. Although ½-in strand can also be used, the 0.6-in strand is more efficient.

14.3.3.2.2 *Tendons*

Tendons are proprietary systems that consist of an anchorage, duct, grout injection pipes and prestressing strand. Smaller tendons used in decks have ducts usually made from HDPE and contain up to four strands. Girder tendons use ducts made from galvanized metal and plastic and usually contain from 12 to 31 strands. The outside diameter of the ducts vary from 3 in to 5 in depending upon the number of strands and system supplier. Consult specific post-tensioning system brochures for the actual size of ducts. Two to five tendons are usually needed for each girder web to satisfy design requirements. The center of gravity specified at anchorages shall be consistent with tendon anchorage requirements (e.g., anticipated size(s) of bearing plates).

For cast-in-place, post-tensioned box girder bridges, tendons are internal to the girder webs. Segmental bridges can have tendons either external or internal to the girder web but not a mixture of the two.

14.4 CAST-IN-PLACE REINFORCED CONCRETE SLABS

14.4.1 General

Reference: LRFD Article 5.14.4

This Section presents information for the design of CIP concrete slabs that amplify or clarify the provisions in the *LRFD Specifications*. The Section also presents design information specific to NDOT practices.

14.4.1.1 Haunches

Haunches at interior supports of continuous bridges allow an increase in span by reducing the maximum positive moment and increasing the negative moment resistance. Parabolic haunches are preferred if aesthetics are important; otherwise, use straight haunches because they are easier to construct. The length of haunch on either side of an interior support should be 15% of the interior span. The depth of haunch at an interior support should be approximately 20% deeper than the structure depth at the location of maximum positive moment.

14.4.1.2 Minimum Reinforcement

Reference: LRFD Articles 5.7.3.3.2, 5.10.8 and 5.14.4.1

In both the longitudinal and transverse directions, at both the top and bottom of the slab, the minimum reinforcement should be determined according to the provisions of LRFD Articles 5.7.3.3.2 and 5.10.8. The first Article is based on the cracking flexural strength of a component, and the second Article reflects requirements for shrinkage and temperature. In CIP concrete slabs, the two Articles provide nearly identical amounts of minimum reinforcement in the majority of cases.

According to LRFD Article 5.14.4.1, the bottom transverse reinforcement (the above minimum provisions notwithstanding) may be determined either by two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with LRFD Equation 5.14.4.1-1. The span length, L, in the equation should be taken as that measured from the centerline to centerline of the supports. For bridges with a skew greater than 60° and/or horizontally curved bridges, the analytical approach is recommended.

Section 14.4.5 presents a simplified approach for shrinkage and temperature steel requirements.

14.4.2 Allowance for Dead-Load Deflection and Settlement

Reference: LRFD Article 5.7.3.6.2

In setting falsework for CIP concrete slabs, an allowance shall be made for the deflection of the falsework, for any settlement of the falsework, for the dead-load deflection of the span, and for the long-term dead-load deflection of the span such that, on removal of the falsework, the top of the structure shall conform to the theoretical finished grade plus the allowance for long-term deflection.

14.4.3 Construction Joints

Longitudinal construction joints on CIP concrete slab bridges are undesirable. However, bridge width, staged construction, the method of placing concrete, rate of delivery of concrete, and the type of finishing machine used by the contractor dictate whether or not a CIP concrete slab bridge must be poured in one or more pours.

If the slab will be built in stages, show the entire lap splice for all transverse reinforcing steel on the side of the construction joint that will be poured last.

14.4.4 Longitudinal Edge Beam Design

Reference: LRFD Articles 5.14.4.1, 9.7.1.4, and 4.6.2.1.4

Edge beams must be provided along the edges of CIP concrete slabs. Structurally continuous barriers may only be considered effective for the Service limit states, not the Strength or Extreme-Event limit states. The edge beams shall consist of more heavily reinforced sections of the slab. The width of the edge beams may be taken to be the width of the equivalent strip as specified in LRFD Article 4.6.2.1.4b.

14.4.5 Shrinkage and Temperature Reinforcement

Reference: LRFD Articles 5.6.2 and 5.10.8

NDOT practice is that evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep and movements of supports is not necessary when designing CIP concrete slabs. Figure 14.4-A provides the shrinkage and temperature reinforcement as a function of slab thickness.

14.4.6 Cap Design

NDOT typically uses an integral cap design in conjunction with CIP concrete slabs. However, NDOT allows the use of non-integral drop caps where aesthetics are not an issue.

Slab Thickness	Reinforcement (Top and Bottom)
< 18″	#4 @ 12″
18″ to 28″	#5 @ 12″
> 28″	Design per LRFD Article 5.10.8.2

SHRINKAGE AND TEMPERATURE REINFORCEMENT FOR CIP CONCRETE SLABS

Figure 14.4-A

14.4.7 Distribution of Concrete Barrier Railing Dead Load

The dead load of the barrier shall be assumed to be distributed uniformly over the entire bridge width.

14.4.8 Distribution of Live Load

Reference: LRFD Article 4.6.2.3

The following specifically applies to the distribution of live load to CIP concrete slabs:

1. For continuous slabs with variable span lengths, one equivalent strip width (E) shall be developed using the shortest span length for the value of L_1. This strip width should be used for moments throughout the entire length of the bridge.

2. The equivalent strip width (E) is the transverse width of slab over which an "axle" unit is distributed.

3. Different strip widths are specified for the CIP concrete slab itself and its edge beams in LRFD Articles 4.6.2.3 and 4.6.2.1.4, respectively.

4. In most cases, using LRFD Equation 4.6.2.3-3 for the reduction of moments in skewed slab-type bridges will not significantly change the reinforcing steel requirements. Therefore, for simplicity of design, NDOT does not require the use of the reduction factor "r."

14.4.9 Shear Resistance

Reference: LRFD Article 5.14.4.1

Single-span and continuous-span CIP concrete slabs, designed for moment in conformance with LRFD Article 4.6.2.3, may be considered satisfactory for shear.

14.4.10 Minimum Thickness of Slab

Reference: LRFD Article 2.5.2.6.3

When using the equations in LRFD Table 2.5.2.6.3-1, it is assumed that:

- S is the length of the longest span.
- The calculated thickness includes the ½-in sacrificial wearing surface.
- The thickness used may be greater than the value obtained from the LRFD Table.
- Minimum slab thickness is 18 in.

14.4.11 Development of Flexural Reinforcement

Reference: LRFD Article 5.11.1.2

LRFD Article 5.11.1.2 presents specifications for the portion of the longitudinal positive-moment reinforcement that must be extended beyond the centerline of support. Similarly, LRFD Article

5.11.1.2.3 addresses the location of the anchorage (embedment length) for the longitudinal negative-moment reinforcement.

14.4.12 Skews on CIP Concrete Slabs

Reference: LRFD Article 9.7.1.3

For skew angles up to 20°, the transverse reinforcement typically runs parallel to the skew, providing for equal bar lengths. For skews in excess of 20°, the transverse reinforcement should be placed perpendicular to the centerline of the bridge. This provision concerns the direction of principal tensile stresses, because these stresses develop in heavily skewed structures, and the provision is intended to prevent excessive cracking.

14.4.13 Abutment Type

For CIP concrete slabs, NDOT generally prefers the use of a seat type abutment.

14.5 PRESTRESSED CONCRETE SUPERSTRUCTURES

14.5.1 General

Reference: LRFD Article 5.2

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 two methods of applying the prestressing force, as discussed in the following Sections. A combination of these two methods may be used if approved by the Chief Structures Engineer.

14.5.1.1 "Pretensioning"

In the pretensioning method, tensioning of the steel strands is accomplished before the concrete is placed. 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."

14.5.1.2 "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 several strands, are loaded into ducts cast into the concrete. After stressing the tendons to the specified prestressing level, it is 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.

14.5.1.3 "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.

The problems with partial prestressing include:

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

It is NDOT practice to not use partial prestressing. Although uncommon, NDOT occasionally uses partial prestressing in post-tensioning applications to counteract dead-load creep; e.g., partial prestressing may be used for the widening portion of conventionally reinforced bridges. This allows the widening to be tied into the existing bridge as soon as the post-tensioning is complete without waiting for an additional 60 to 90 days for creep deflection mitigation. However, in these applications, the widening must be designed to resist all forces with the mild reinforcement; the post-tensioning is supplemental and for the bridge dead load only.

14.5.2 Basic Criteria

This discussion applies to both pretensioned and post-tensioned concrete members.

14.5.2.1 Concrete Stress Limits

Reference: LRFD Article 5.9.4

Tensile stress limits for fully prestressed concrete members shall conform to the requirements for "Other Than Segmentally Constructed Bridges" in LRFD Article 5.9.4, except that the tensile stress at the Service limit state, after losses, shall be limited as follows: For components with bonded prestressing tendons or reinforcement, the tensile stress in the precompressed tensile zone shall be limited to:

$$0.095\sqrt{f'_c}$$

14.5.2.2 Concrete Strength at Release

Reference: LRFD Article 5.9.4.1

At release of the prestressing force, the minimum compressive concrete strength shall be the greater of 3.0 ksi or 60% of the specified 28-day strength. The specified concrete compressive strength at release should be rounded to the next highest 0.1 ksi.

14.5.2.3 Loss of Prestress

Reference: LRFD Article 5.9.5

Loss of prestress is defined as the difference between the initial stress in the strands and the effective prestress in the member. This definition of loss of prestress includes both instantaneous and time-dependent losses and gains.

The 2005 interim changes to the *LRFD Specifications* include many revisions to the process of calculating the loss of prestress.

14.5.2.4 Strand Transfer Length and Development Length

Reference: LRFD Article 5.11.4

The transfer length is the length of strand over which the prestress force is transferred to the concrete by bond and friction. The *LRFD Specifications* indicates that the transfer length may be assumed to be 60 strand diameters. The stress in the strand is assumed to vary linearly from zero at the end of the member, or the point where the strand is bonded if debonding is used, to the full effective prestress force at the end of the transfer length.

The development length is the length of strand required to develop the stress in the strand corresponding to the full flexural strength of the member; i.e., strand development length is the length required for the bond to develop the strand tension at nominal flexural resistance. The transfer length is included as part of the development length. LRFD Equation 5.11.4.2-1 is used to calculate the required development length (l_d). Prestressing strands shall be considered fully bonded beyond the critical section for development length. The development length for debonded strands shall be in accordance with LRFD Article 5.11.4.3.

14.5.2.5 Skew

Reference: LRFD Article 4.6.2.2

The behavior of skewed bridges is different from those of rectangular layout. The differences are largely proportional to skew angle. Although normal flexural effects due to live load tend to decrease as the skew angle increases, shear does not, and there is a considerable redistribution of shear forces in the end zone due to the development of negative moments therein. For skew angles less than 20°, it is considered satisfactory to ignore the effects of skew and to analyze the bridge as a straight bridge.

LRFD Articles 4.6.2.2.2e and 4.6.2.2.3c provide tabulated assistance to roughly estimate the live-load effects from skew. The factors shown in these tables can be applied to both simple span and continuous span skewed bridges. The correction factors for shear theoretically only apply to support shears of the exterior girder at the obtuse corner. In practice, the end shears of all girders in a multi-girder bridge are conservatively modified by the skew correction factor. Shear in portions of the girder away from the end supports do not need to be corrected for skew effects.

To obtain a better assessment of skewed behavior and to use potential benefits in reduced live-load moments, more sophisticated methods of analysis are used. The refined methods most often used to study the behavior of bridges are the grillage analysis and the finite element method. See Section 13.2 for more discussion. The finite element analysis requires the fewest simplifying assumptions in accounting for the greatest number of variables that govern the structural response of the bridge. However, input preparation time and derivation of overall forces for the composite girder are usually quite tedious. On the other hand, data preparation for the grillage method is simpler, and the integration of stresses is not needed.

14.5.3 <u>Cast-in-Place, Post-Tensioned Box Girders</u>

Reference: LRFD Articles 5.4.5 and 5.4.6

14.5.3.1 Ducts

In post-tensioned construction, ducts are cast into the concrete to permit placement and stressing of the tendons. Girder ducts are typically galvanized corrugated steel (semi-rigid). Ducts in top slabs are typically high-density polyethylene. For external tendons on segmental bridges, NDOT typically uses smooth polyethylene. The contract documents shall indicate the type of duct material to be used.

The wall thickness shall be no less than 28 gage. Prebending of ducts will be required for bend radii less than 30 ft and should be specified in the contract documents. Radii that require prebending should be avoided whenever possible. The minimum bend radius of ducts shall not be less than 20 ft, except in anchorage zones where 12 ft will be permitted. The bending radius of polyethylene or polypropylene ducts shall not be less than 30 ft.

If the bridge is constructed by post-tensioning precast components together longitudinally and/or transversely by use of a cast-in-place concrete joint, then the end of the duct should be extended beyond the concrete interface by not less than 3 in and not more than 6 in to facilitate joining the ducts. If necessary, the extension could be in a local blockout at the concrete interface. Joints between sections of ducts shall be positive metallic connections, which do not result in angle changes at the joints. Waterproof tape shall be used at all connections.

For multiple-strand tendons, the outside diameter of the duct shall be no more than 40% of the least gross concrete thickness at the location of the duct. The majority of bridges use a 12-in web, which limits the outside duct diameter to 4.8 in. Larger ducts may be required for shallow bridges with high P-jack forces (more than 1200 kips per girder). A wider web of 14 in may be used to ensure that the post-tensioning system fits. During design, the bridge designer must lay out an acceptable duct arrangement that matches the post-tensioning center of gravity to determine if a wider web is needed. Ducts preferably should not extend into either the top or bottom slabs. The internal free area of the duct shall be at least 2.5 times the net area of the prestressing steel. See LRFD Article 5.4.6.2.

Section 503 of the *NDOT Standard Specifications* discusses ducts for post-tensioned construction.

14.5.3.2 Grouting

Upon completion of post-tensioning, the ducts must be grouted. The strength of the grout should be comparable to that of the girder concrete but is not specified due to the high strengths that typically result from tendon grouts.

NDOT requires pre-approved bagged grout for tendon grouting. Multiple injection and bleed ports are required at the ends of the tendons and at all low and high points. Flushing of tendons due to blockage is discouraged but not disallowed using vacuum grouting as a consideration for repairs. Drilling into a percentage of the tendons at the anchorage to inspect for voids is a requirement of the *NDOT Standard Specifications*. If any voids are found, all tendons are inspected.

14.5.3.3 Tendon Profile

The geometry of a typical tendon profile is predominantly composed of second-degree parabola curved segments. The tendons are essentially straight segments near the anchorages. The tendon group center of gravity and the bridge's neutral axis should coincide at the following locations at the centerlines of abutments, hinges and points of dead-load contraflexure.

Show offset dimensions to post-tensioning duct profiles from fixed surfaces or clearly defined reference lines. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Curved ducts that run parallel to each other, ducts in curved girders, ducts in chorded girders where angle changes occur between segments, or ducts placed around a void or re-entrant corner shall be sufficiently encased in concrete and reinforced as necessary to avoid radial failure (pull-out into the other duct or void).

14.5.3.4 Anchorages

There are several types of commercially available anchorages. These anchorages normally consist of a steel block with holes in which the strands are individually anchored by wedges. In the vicinity of the anchor block (or coupler), the strands are fanned out to accommodate the anchorage hardware. The fanned out portion of the tendon is housed in a transition shield, often called a trumpet, which could be either steel or polyethylene, regardless of the duct material. Trumpets must have a smooth, tangential transition to the ducts.

If the distance between anchorages exceeds 300 ft, jacking at both ends should be considered. One-end or two-end stressing will be determined by design and specified in the contract documents.

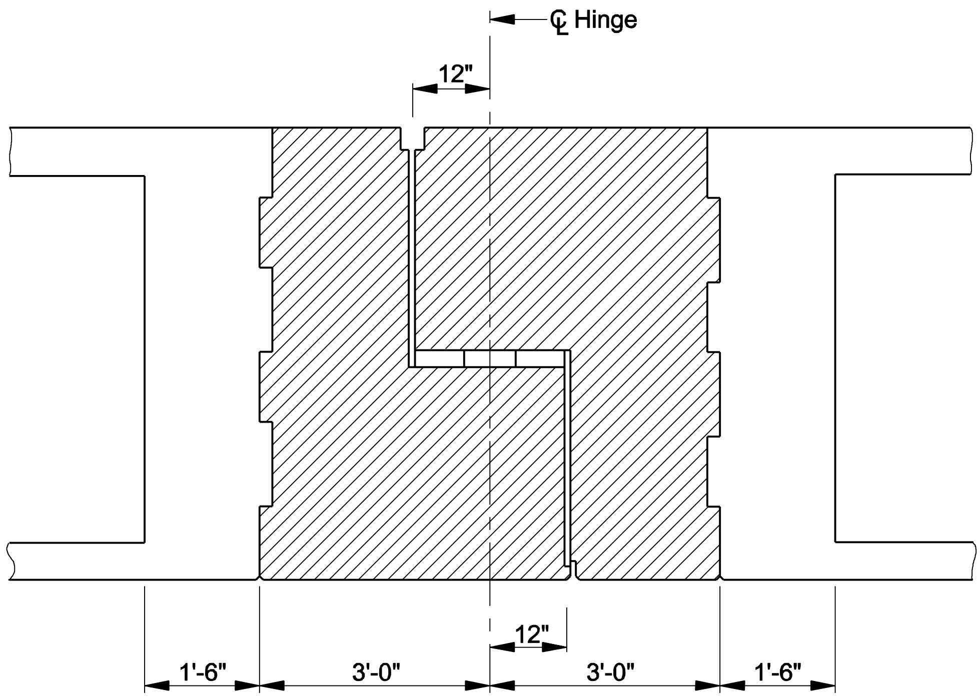
Values of the wobble and curvature friction coefficients and the anchor set loss assumed for the design shall be shown in the contract documents.

14.5.3.5 Hinges

For cast-in-place, post-tensioned, concrete superstructures, an in-span hinge is a complicated element that requires special consideration related to its design, detailing and construction sequencing. Intermediate expansion joints are often introduced into the superstructure of longer bridges, thereby dividing the structure into shorter frames with the intention of reducing thermal, creep and shrinkage forces in outlying supports. It is preferable to locate expansion joints atop intermediate piers, presuming there is adequate vertical clearance to accommodate a drop cap and bearing seat. Where a drop cap is not feasible nor aesthetically desirable, it may be necessary to introduce an in-span hinge.

Figure 14.5-A illustrates a longitudinal cross section of a typical in-span hinge for a cast-in-place, post-tensioned concrete box girder. As noted, the cross hatched area indicates the portion of the hinge concrete that is cast after the supporting (short cantilever) and supported spans have been post-tensioned. The supporting and supported sides of the hinge shall be designed as corbel elements and confirmed with a strut-and-tie approach to establish an adequate load path through the hinge.

The effects of post-tensioning in both the supporting and supported spans shall be considered in the design and detailing. For example, when the supported span is tensioned prior to hinge casting, the dead load will be redistributed from interior falsework supports to the temporary bent beneath the hinge end of the span. The contract documents should include the dead load reaction at the hinge for the contractor's use in the design of this temporary support.

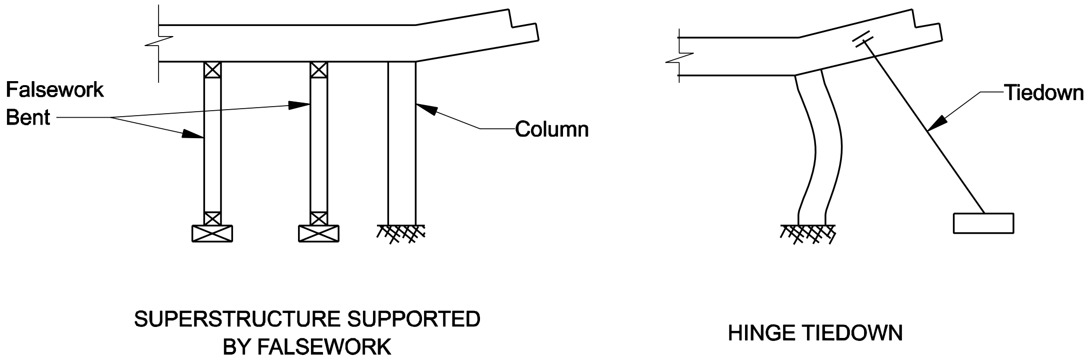


LONGITUDINAL CROSS SECTION OF IN-SPAN HINGE

Figure 14.5-A

After the supporting span is tensioned but before the load from the supported span has been transferred through the hinge, there is a tendency for the unloaded short cantilever to deflect upward. This phenomenon is commonly termed “hinge curl” and will increase over time with concrete creep and shrinkage. Unless properly accounted for, hinge curl can negatively impact the final deck profile and may result in the need for extensive grinding to correct the problem.

The designer is reminded that there is a variable period of time (usually between 30 and 180 days, occasionally much longer) in which the short cantilever remains unloaded after it has been stressed. The period of time and, therefore, the extent of hinge curl are not predictable until the contractor’s schedule is known. Furthermore, experience indicates that the hinge does not always deflect downward at load transfer as much as it had previously deflected upward under the influence of the prestressing force. The procedure presented below to address hinge curl assumes that the falsework in the adjacent back span will remain in place until load transfer occurs at the hinge. The back span falsework will, to some extent, assist in resisting the column top rotation that can contribute further to the upward movement of the short cantilever (see Figure14.5-B). Where it is necessary to remove back span falsework prior to hinge load transfer, the procedure identified below is not sufficient, and the designer should consider tying down the short cantilever or preloading it with temporary weights placed on the deck.



HINGE CURL COUNTERMEASURE

Figure 14.5-B

Notations and nomenclature:

e = eccentricity of prestressing at hinge of short cantilever
E = concrete modulus of elasticity
I = moment of inertia of box girder section
L = length of short cantilever
P_h = horizontal component of Pjack at anchorage of short cantilever
P_v = vertical component of Pjack as anchorage of short cantilever
T = transfer load at short cantilever
w = unit dead load of short cantilever

Deflection of short cantilever:

$$\Delta_{curl} = -(P_vL^3)/(3EI) \;\; +/- \;\; (P_heL^2)/2EI \;+\; wL^4/8EI \;+\; PL/3EI$$

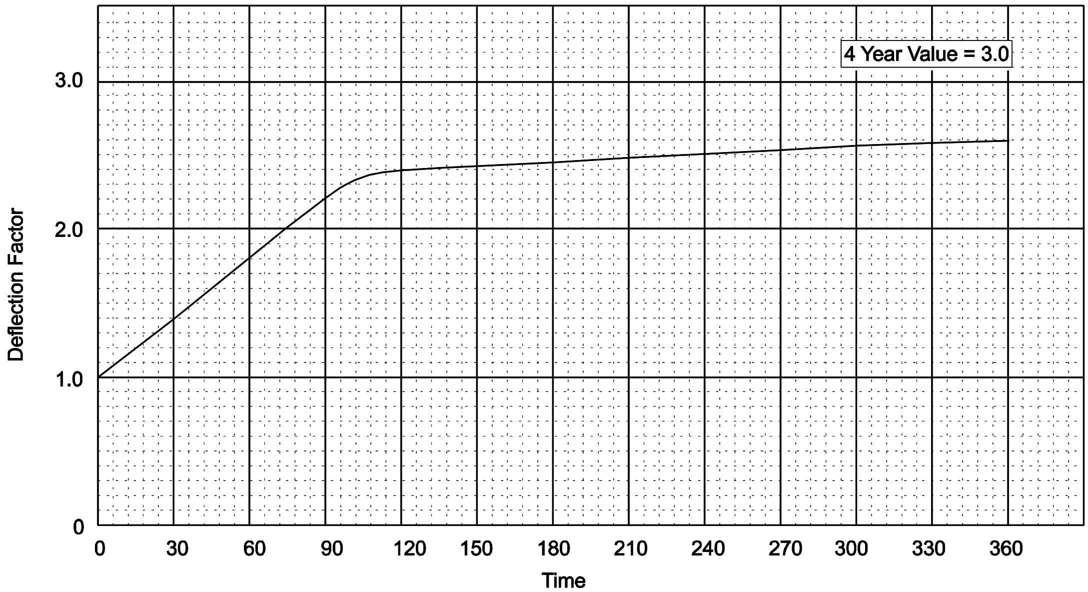
(Positive indicates upward deflection)

Deflection components:

$(P_vL^3)/(3EI)$ = prestressing vertical component
$(P_heL^2)/2EI$ = prestressing horizontal component
$wL^4/8EI$ = distributed dead load of superstructure without barrier, future overlay, etc.
$PL/3EI$ = concentrated load from hinge diaphragm or other miscellaneous load

Downward deflection of short cantilever:

$$\Delta_{reaction} = TL^3/3EI$$



TIME vs. DEFLECTION

Figure 14.5-C

Modify deflections for long-term effects using the Time vs. Deflection chart shown in Figure 14.5-C and as follows:

1. **Adjustment "A"**. Profile adjustment required at the long cantilever for transfer dead load less prestress uplift after load transfer (may be positive or negative value):

   30-day value $= 2.60 \times \Delta_{reaction} - 1.60 \times \Delta_{curl}$
   60-day value $= 2.20 \times \Delta_{reaction} - 1.20 \times \Delta_{curl}$
   90-day value $= 1.80 \times \Delta_{reaction} - 0.80 \times \Delta_{curl}$
   120-day value $= 1.60 \times \Delta_{reaction} - 0.60 \times \Delta_{curl}$
   180-day value $= 1.55 \times \Delta_{reaction} - 0.55 \times \Delta_{curl}$
   240-day value $= 1.50 \times \Delta_{reaction} - 0.50 \times \Delta_{curl}$
   360-day value $= 1.40 \times \Delta_{reaction} - 0.40 \times \Delta_{curl}$
   720-day value $= 1.25 \times \Delta_{reaction} - 0.25 \times \Delta_{curl}$

2. **Adjustment "B"**. Profile adjustment required for short cantilever (may be positive or negative value):

   30-day value $= 2.60 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   60-day value $= 2.20 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   90-day value $= 1.80 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   120-day value $= 1.60 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   180-day value $= 1.55 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   240-day value $= 1.50 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   360-day value $= 1.40 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   720-day value $= 1.25 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$

The predicted deflections shall be incorporated into the camber diagram that is included in the contract documents. It is assumed that the long-term effect of creep and shrinkage will result in an ultimate deflection three times greater than the theoretical immediate deflection, and that this will occur over a four-year period. Because the transfer of load from the supported span will occur at an unknown time after prestressing the supporting span, a camber diagram with time-dependent tabulated values shall be shown in the contract documents to account for schedule uncertainty.

Figures 14.5-D and 14.5-E illustrate the camber diagram to be drawn for a hinged span. The normal camber diagram is shown along with an enlarged camber curve for the hinged span. Values of camber are calculated and shown at the hinge. Adjustment "A" is calculated for the position of the supported span hinge side, and Adjustment "B" is calculated for the position of the supporting span hinge side. Point 1 in Figure 14.5-F is the theoretical camber if load transfer could be immediate from supported to supporting span (short cantilever). Point 2 is the adjustment (up or down) to the theoretical camber for the supported span, which is dependent on the time of transfer. Point 3 is the adjustment (up or down) to the theoretical camber for the supporting span (short cantilever).

14.5.3.6 Flexural Resistance

Reference: LRFD Article 5.7.3.2

Flexural resistance for CIP P/T concrete box girders shall be determined using the combined effects of bonded prestressing and mild reinforcing steel in accordance with LRFD Article 5.7.3.2.

14.5.3.7 Shear Resistance

14.5.3.7.1 *Strength Limit State*

Reference: LRFD Article 5.8.3

The shear resistance of CIP, PT boxes shall be determined using the modified compression field theory (MCFT) sectional model of LRFD Article 5.8.3.4.2.

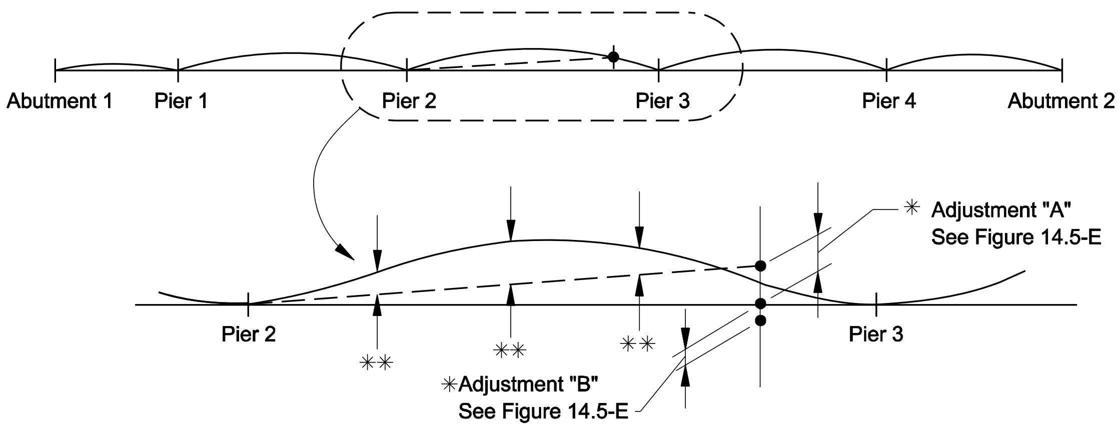
14.5.3.7.2 *Service Limit State*

Reference: LRFD Article 5.8.5

The principal stress-limit requirements of LRFD Article 5.8.5 shall apply to CIP, PT boxes at the Service limit state.

14.5.3.8 Falsework

Cast-in-place, post-tensioned box girder bridges must be supported during their construction. They cannot support even their own dead load until post-tensioning is complete. The temporary supports used are either earth fills, if traffic does not have to be maintained, or falsework. Earth fills must be compacted sufficiently to keep settlement to a minimum. Falsework usually consists of a combination of timber and steel structural components. The falsework is designed to carry the entire dead load of the bridge and construction loads in accordance with the *NDOT Standard Specifications.*



Notes:

✳ See Figure 14.5-E for values to use. These depend on period of time between prestressing of Frame 2 and load transfer from Frame 1.

✳ ✳ Adjusted values of camber taken from long cantilever values from analysis. Decrease linearly back to pier for profile adjustment (add to profile grade).

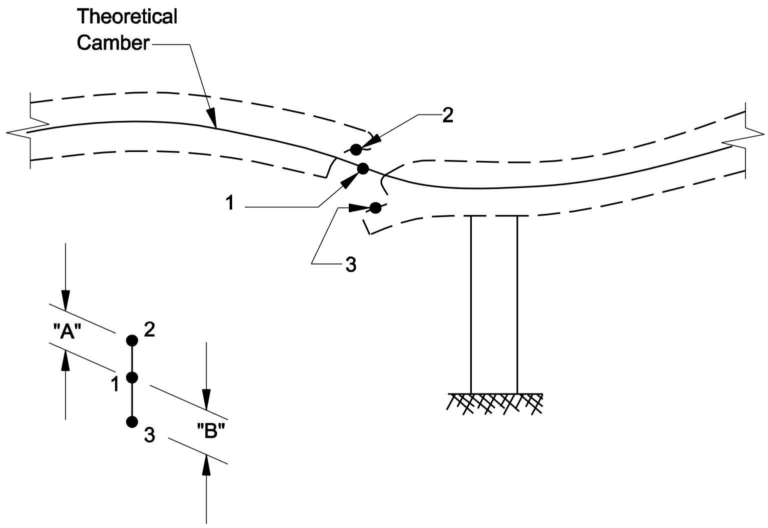
CAMBER DIAGRAM

Figure 14.5-D

Elapsed time in days measured from prestressing short hinge side until closure and load transfer	Adjustment "A"	Adjustment "B"
30 days		
60 days		
90 days		
120 days		
180 days		
240 days		
360 days		
720 days		

TIME-DEPENDENT CAMBER VALUES

Figure 14.5-E



Note: Instead of a field adjustment from theoretical camber as shown above, use Figure 14.5-D and Figure 14.5-E for a direct adjustment to the profile grade.

ADJUSTMENT TO THEORETICAL CAMBER FOR HINGED SPAN

Figure 14.5-F

The contractor submits falsework calculations and shop drawings for review and approval. A Nevada registered professional civil/structural engineer must prepare and stamp the shop drawings. In addition, the registered engineer must inspect the completed falsework and certify that it was built according to the approved falsework drawings.

14.5.3.9 Diaphragms

At a minimum, intermediate diaphragms must be placed at mid-span of CIP concrete box girder superstructures. For longer spans, particularly on curved alignment, additional diaphragms should be considered to enhance the distribution of load among girder webs.

14.5.3.10 Responsibilities (Designer/Contractor)

For CIP, post-tensioned concrete box girder bridges, the designer is responsible for establishing the profile for the center of gravity of the post-tensioning steel (see Section 14.5.3.3) and for defining the total jacking force (Pjack) to be applied to the superstructure. The contractor (usually a specialty subcontractor) will determine the number of strands and tendons that will be supplied in each girder in accordance with the requirements for distribution of prestressing force and stressing sequence that is defined in Standard Plan B-28.1.1 and the *Standard Specifications*.

The contractor is required to submit shop drawings that define the details of the proposed post-tensioning system (including the number of tendons per girder, number of strands per tendon, tendon duct layout, anchorage devices, stressing sequence, jacking force for each tendon and theoretical tendon elongation). The bridge designer will review the shop drawings to confirm that the contractor's system provides the correct center of gravity, that the required total jacking force has been provided, and that the requirements for distribution and sequence of stressing have been satisfied. See Appendix 25A.

Post-tensioning systems require confinement reinforcement to distribute the large concentrated forces. The contractor is responsible for the "local zone" reinforcement. See LRFD Article 5.10.9.2.3. This reinforcing steel controls the concrete cracking around the post-tensioning head and is specific to proprietary post-tensioning systems. This reinforcing steel is determined by the prequalification testing and must be included in the shop drawings. The designer is responsible for the "general zone" reinforcement. See LRFD Article 5.10.9.2.2.

14.5.4 Precast, Prestressed Concrete Girders

14.5.4.1 Precast I-Girder Sections

The type of girders used in the superstructure are selected based upon geometric restraints, economy and appearance. NDOT has not adopted standard precast concrete I-girder sections. PCI has developed standard sections that are used in most locations throughout the United States. However, precasting plants in States adjacent to Nevada may use standard sections specific to their State. Because there are no precasting plants in Nevada, the designer should contact precasters that are likely to provide girders for a project and discuss their product line.

To ensure that the structural system has an adequate level of redundancy, NDOT requires a minimum of four girder lines on new bridges, except as allowed in Section 11.4.5.2 of this *Manual*.

14.5.4.2 General

Reference: LRFD Article 5.9

This Section addresses the general design theory and procedure for precast, prestressed (pre-tensioned) concrete girders. For design examples, consult the *PCI Bridge Design Manual*, Chapter 9.

Bridges consisting of simple-span precast concrete girders and cast-in-place concrete slabs shall be made 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, the member is treated as an ordinary reinforced concrete section, and the bottom flanges of adjoining girders are connected at the interior supports by reinforcement projecting from girder ends into a common diaphragm. The members shall be assumed to be fully continuous with a constant moment of inertia when determining both the positive and negative moments due to loads applied after continuity is established.

The resistance factor "φ" (LRFD Article 5.5.4) for flexure shall be 1.0, except for the design of the negative-moment steel in the deck for structures made continuous for composite loads only and having a poured-in-place continuity diaphragm between the ends of the girders over the piers. For this case, the resistance factor φ shall be the 0.90 value for reinforced concrete members in flexure.

14.5.4.3 Stage Loading

There are four loading conditions that must be considered in the design of a precast, prestressed girder:

1. The first loading condition is when the strands are tensioned 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. It is the fabricator's responsibility to consider these factors during the fabrication of the girder and to make adjustments to 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 ("hog 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 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. At the designer's option, if he/she chooses to consider the transfer length of the strands at the end of the girder and at the debonding points, then the stress in the strands should be assumed to be 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:

   - 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;

   - 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; or

   - deviating some of the strands to reduce the strand eccentricity at the end of the girder.

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

3. The third loading condition occurs several weeks to several months after strand release when the girder is erected and the composite deck is 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. Reliable estimates of deflection and camber are needed 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.

4. The fourth 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.

14.5.4.4 Debonded Strands

Debonding of strands at the ends of precast, pretensioned concrete girders will be allowed on projects for NDOT with the following restrictions:

1. A maximum of 25% of the total number of prestressing strands may be debonded to satisfy the allowable stress limits. In any row, debonded strands shall not exceed 40% of the total strands in that row.

2. Not more than 40% of the debonded strands or four strands, whichever is greater, shall be terminated at any section.

3. Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the girder.

4. The theoretical number of debonded strands shall be rounded to the closest even number (pairs) of strands, except that debonded strands will not be permitted in rows containing three strands or less.

5. All exterior strands shall be fully bonded (including the entire bottom row).

6. At each end of a girder, the maximum length for debonding is 15% of the entire girder length.

In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the values in LRFD Article 5.9.4, except that tension is limited to 0.0948 $\sqrt{f'_c}$ for all exposure conditions.

14.5.4.5 Flexural Resistance

The design of prestressed concrete members in flexure normally begins with the determination of the required prestressing level to satisfy service conditions. All load stages that may be critical during the life of the structure from the time prestressing is first applied should be considered. This is then followed 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.

For checking the stresses in the girder at the Service limit state, the following basic assumptions are made:

1. Planar sections remain plane, and strains vary linearly over the entire member depth. Therefore, composite members consisting of precast concrete girders and cast-in-place decks must be adequately connected so that this assumption is valid and all elements respond to superimposed loads as one unit. Deck concrete is transformed to girder concrete when computing section properties by multiplying the effective deck width by

the ratio of the deck concrete modulus of elasticity to the girder concrete modulus of elasticity. The gross concrete section properties shall be used (i.e., the area of prestressing strands and reinforcing steel is not transformed).

2. The girder is assumed to be uncracked at the Service limit state.

3. Stress limits are not checked for the deck concrete in the negative-moment region because the deck concrete is not prestressed.

14.5.4.6 Interface Shear

Reference: LRFD Article 5.8.4

Cast-in-place concrete decks designed to act compositely with precast concrete girders must be able to resist the interface shearing forces between the two elements. The following formula, substituting LRFD Equation 5.8.4.2-2 into LRFD Equation 5.8.4.2-1, may be used to determine the factored interface shear stress, V_{vi} :

$$V_{ui} = 12\, V_{u1} / d_v$$

The factored interface shear force shall be less than or equal to the factored nominal interface shear resistance; i.e.:

$$V_{ui} \le \phi V_{ni}$$

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

The permanent net force normal to the interface, P_c, may be conservatively neglected if it is compressive.

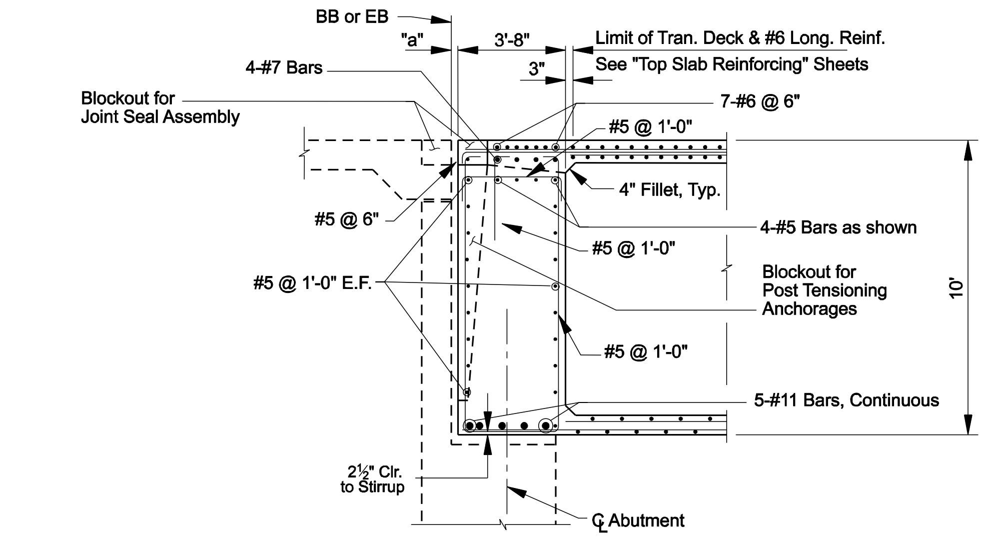
14.5.4.7 Diaphragms

Reference: LRFD Article 5.13.2.2

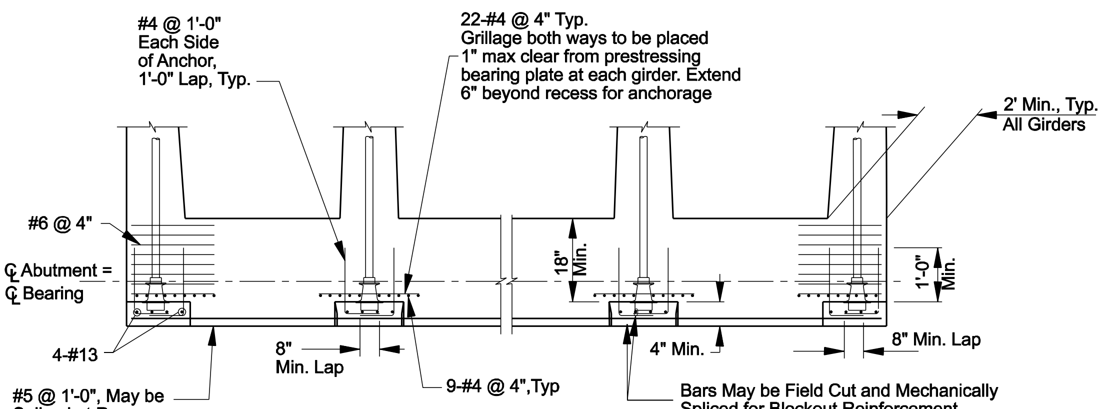
NDOT practice is to place one full-depth, cast-in-place diaphragm between every girder at mid-span. Provide additional diaphragms as required.

For precast, prestressed girder spans, cast-in-place concrete diaphragms shall be used at all supports with the girders embedded a minimum of 6 in into the diaphragm. For spans greater than 40 ft, intermediate diaphragms shall also be used and shall be constructed of cast-in-place concrete. At a minimum, one line of intermediate diaphragms shall be used in each span greater than 40 ft. For skews of 20° or less, the intermediate diaphragms may be placed along the skew of the bridge. For skews in excess of 20°, the intermediate diaphragms shall be placed perpendicular to the girders. The tops of the intermediate diaphragms should be detailed continuous with the deck slab. Slabs shall not be poured until a minimum of seven days after the interior diaphragms are poured or until the diaphragm concrete reaches a compressive strength of 3 ksi.

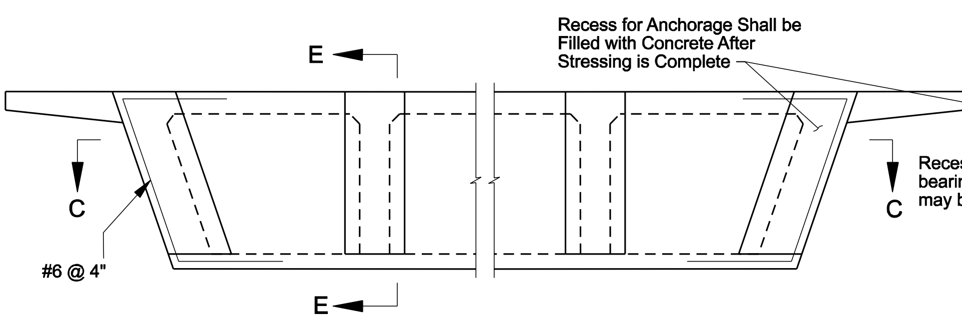
For continuous precast, prestressed girder spans, the closure diaphragms at the piers shall be cast separately from the deck slab. For integral abutments, the end diaphragms shall also be cast separately from the deck slab.



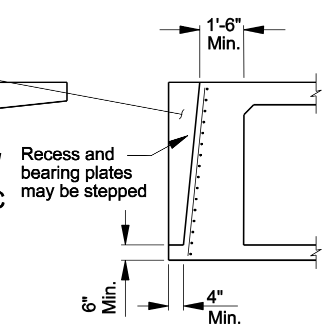
CROSS SECTION OF END DIAPHRAGM



SECTION C-C



PRESTRESS ANCHORAGE DETAILS



SECTION E-E

Note: Dimension "a" varies from project to project.

GIRDER-END DIAPHRAGM DETAIL
(With Recessed Anchorages)

Figure 14.5-G