NDOT STRUCTURES MANUAL
REVISION 2019-4
March 2019

Revision Summary

Page(s)	Manual Subsection	Description
14-1	14.1.1	Correct LRFD references and terminology.
14-1,2	14.1.2	Correct LRFD references and terminology.
14-2	14.1.2.2	Delete additional requirements within subsection. Specify minimum reinforcement as required by LRFD.
14-2	14.1.2.3	Revise additional reinforcement in girder webs.
14-3,4	14.1.2.5	Correct LRFD references, remove text and equations provided in LRFD, remove old procedure, clarify LRFD requirement at indirect supports, change shear friction requirement to match LRFD.
14-6	14.1.4	Remove text and equations provided in LRFD.
14-8	14.2.2	Permit the use of higher strength reinforcement in drilled shafts if approved by the Chief Structures Engineer.
14-9	14.3.1.2	Correct LRFD reference.
14-10	Figure 14.3-B	Correct diameter of drilled shafts to 5’.

Revisions indicated by underscored text.

14.1.1 Member Design Models

Reference: LRFD Articles 5.5.1.2, 5.7, and 5.8

Where it is reasonable to assume that a planar section remains planar after loading, the *LRFD Specifications* specifies 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 B-Regions such as the design of typical bridge girders, pier caps, 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 5.5.1.2 discusses the sectional design model. Articles 5.7.1 describes 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 for D-Regions such as regions near discontinuities (e.g., abrupt changes in cross section, openings, coped (dapped) ends, deep girders and caps, corbels). See LRFD Articles 5.5.1.2 and 5.8.

The following Sections discuss each of these member design approaches.

14.1.2 Sectional Design Model

Reference: LRFD Article 5.5.1.2

14.1.2.1 Flexural Resistance

Reference: LRFD Article 5.6

The flexural resistance of a girder section is typically obtained using the rectangular stress distribution with properties described in LRFD Article 5.6.2.2. In lieu of using this simplified, yet accurate approach, a strain compatibility approach may be used as outlined in LRFD Article 5.6.3.2.5. The general equation for structural concrete flexural resistance of LRFD Article 5.6.3.2.2 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 5.5.4.2 and 5.6.3.2

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 or 5.5.4.2-2 is applied to differentiate between tension- and compression-controlled sections.

14.1.2.2.2 *Minimum*

Reference: LRFD Articles 5.4.2.6 and 5.6.3.3

14.1.2.3 Distribution of Reinforcement

Reference: LRFD Article 5.6.7

In addition to the provisions of LRFD Article 5.6.7, 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 5.6.7. 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 webs of cast in place box girders a minimum of 0.8 in^2 reinforcement shall be located within 3 inches of the construction joint at the top of each web, and in the negative moment regions a minimum of 1.5 in^2 shall be provided at the top of each web.

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 5.6.7

Reinforcing bars in all reinforced concrete members in tension shall be distributed to control cracking in accordance with LRFD Article 5.6.7. 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 5.7

14.1.2.5.1 *Sectional Design Models*

Reference: LRFD Article 5.7.3

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* no longer allow the simplified procedure (V_{ci} and V_{cw} method) found in the AASHTO Standard Specifications for Highway Bridges for estimating the shear resistance of concrete members.

Article 5.7.3.5 addresses the additional tension due to shear that must be resisted by the longitudinal reinforcement. The longitudinal reinforcement required by equation 5.7.3.5-1 in continuous beams with "indirect supports" need not exceed the amount required to resist flexure due to the maximum moment.

14.1.2.5.2 *Shear Friction*

Reference: LRFD Article 5.7.4

All reinforcement fully developed across a plane may be considered for interface shear in accordance with the provisions of LRFD Article 5.7.4.

14.1.4 Fatigue

Reference: LRFD Articles 3.4.1, 3.6.1.4 and 5.5.3

14.1.5 Torsion

Reference: LRFD Article 5.7

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 5.7.2 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; and
- abutment caps, if they are unsymmetrically loaded.

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 longitudinal reinforcement in capacity protected drilled shafts and 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.

14.3.1.2 Concrete Cover

Reference: LRFD Article 5.10.1

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.

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 ≥ 5′)		6″
Drilled Shafts (Diameter < 5′)		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