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Chapter Sixty-two

REINFORCED CONCRETE

Section 5 of the *LRFD Bridge Design Specifications* specifies the design requirements of concrete in all structural elements. This Chapter presents supplementary information specifically on the general properties of concrete and reinforcing steel and the design of reinforced concrete. Chapter Sixty-three discusses prestressed concrete superstructures.

References shown following section titles are to the AASHTO LRFD *Bridge Design Specifications.*

62-1.0 GENERAL DESIGN CONSIDERATIONS

62-1.01 Introduction/Material Properties

Reference: Article 5.4

The minimum yield strength for reinforcing steel should be taken as 420 MPa.

Figure 62-1A presents criteria for concrete materials in structural elements.

62-1.02 Flexure

Reference: Article 5.7

The flexural response of a beam section is obtained on the basis of its compatibility and equilibrium. Compatibility means that the stress-strain relationship for both steel and concrete follow a predetermined course. Once the steel yields, however, the relationship becomes undetermined. Equilibrium means that the sum of internal force effects is equal to the outside force effects.

To facilitate design, the *LRFD Specifications* provides a simplified sectional stress distribution for the strength limit state, the application of which is limited to under-reinforced rectangular sections. Stresses in both top and bottom steels are taken at yield, while the concrete stress block is assumed to be rectangular with an intensity of $0.85f_c'$ and a height as described by the equation as follows:

$$a = \frac{A_s f_y - A'_s f'_y}{0.85 f'_c\, b}$$

Location of the neutral axis is calculated as follows:

$$c = a \div \beta_1$$

The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 28 MPa. For concrete strengths exceeding 28 MPa, β_1 shall be reduced at a rate of 0.05 for each 7 MPa of strength in excess of 28 MPa. However, β_1 shall not be taken to be less than 0.65, per LRFD Article 5.7.2.2, and the nominal flexural resistance as follows:

$$M_n = A_s f_y\,[d_s - 0.5a] - A'_s\, f'_y\,(d'_s - 0.5a)$$

62-1.03 Limits for Reinforcing Steel

Reference: Article 5.7.3.3

62-1.03(01) Maximum

Article 5.7.3.3.1 of the *LRFD Specifications* regulates the maximum allowable steel for reinforced and prestressed members by limiting the c/d_e ratio to 0.42. The *LRFD Specifications* forbids over-reinforced concrete components due to the following:

1. An inelastic mechanism controlled by the yield of steel provides more ductility.

2. At strength limit state, the damage to the concrete is more irreversible with over-reinforced components.

3. Obtaining a reliable estimate of flexural strength of an over-reinforced component requires a full-scale analysis of the sectional behavior using an inelastic concrete stress/strain relationship. The nominal strength as provided by Equation C5.7.3.3.1-1 in the Commentary can only be considered as a conservative approximate figure.

62-1.03(02) Minimum

In accordance with LRFD Article 5.7.3.3.2, the minimum reinforcement should be checked at any section to make sure that the amount of prestressed and non-prestressed reinforcement is enough to develop a factored flexural resistance (M_r) at least equal to the lesser of at least 1.2 times the cracking moment (M_{cr}) or 1.33 times the factored moment required by the applicable strength load combinations. Most often, $1.2M_{cr}$ controls in the maximum positive moment regions. In the region located approximately within the end one-third of the beam or span, 1.33 times the factored moment will generally control.

Use Equation 5.7.3.6.2-2 to compute the cracking moment.

$$M_{cr} = \frac{f_r I_g}{y_t}$$

Where:

M_{cr} = cracking moment (N-mm)
f_r = modulus of rupture of concrete as specified in Article 5.4.2.6 (MPa)
y_t = distance from the neutral axis to the extreme tension fiber (mm)

For a rectangular section (ignoring compression reinforcement), use the equation as follows:

$$M_{cr} = f_r \frac{I_g}{y_t}$$

The factored resistance is as follows:

$$M_r = 0.9M_n$$

Accordingly,

$$1.2M_{cr} \le 0.9A_s f_y \; d\left[1.0 - \frac{A_s f_y}{1.7bdf_c'}\right]$$

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Example 62-1.1

For b = 305 mm, h = 203 mm and f_c' = 28 MPa:

$$M_{cr} = (0.105)(305)(203^2)\sqrt{28} = 6983 \text{ x } 10^3 \text{ kN-mm},$$

d = 171 mm, and f_y = 420 MPa.

$$B = \frac{(-1.7)(bdf_c')}{f_y} = \frac{(-1.7)(305)(171)(28)}{420} = -5911 \text{ mm}^2$$

$$C = \frac{2.267 M_{cr} b f_c'}{f_y^2} = \frac{(2.267)(6983 \ x \ 10^3)(305)(28)}{420^2} = 766\ 000 \ mm^4$$

from which,

$$A_s = 0.5\left[-B - \sqrt{B^2 - 4C} \ \right] = 133 \ mm^2 \qquad \text{(Equation 62-1.1)}$$

or a ratio of $\rho = 133 \div (305 \text{ x } 203) = 0.002\ 15$

This process also provides the minimum steel in both directions at the top and bottom of a concrete slab bridge.

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62-1.04 Shear and Torsion

Reference: Article 5.8

The *LRFD Specifications* allows two methods of shear design for prestressed concrete, the strut-and-tie model and the 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 beam theory are valid. This theory 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, but it does not consider the specific details of how the force effects were introduced into the member.

In regions near discontinuities, such as abrupt changes in cross-section, openings, coped (dapped) ends, deep beams, and corbels, the strut-and-tie model should be used. See LRFD Articles 5.6.3 and 5.13.2.

LRFD Article 5.8.3 discusses the sectional design model. Subsections 1 and 2 describe the applicable geometry required to use this technique to design web reinforcement.

The nominal resistance is taken as the lesser of the following:

$$V_n = V_c + V_s + V_p, \text{ or} \qquad \text{(LRFD Eq. 5.8.3.3-1)}$$

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

For a non-prestressed section, $V_p = 0$.

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

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

$$V_c = 0.083\beta\sqrt{f'_c}\, b_v d_v \qquad \text{(LRFD Eq. 5.8.3.3-3)}$$

The contribution of the web reinforcement is computed as follows:

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

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

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

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

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. A detailed methodology along with the design tables are provided in LRFD Article 5.8.3.4.2. For a section 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 of the *Specifications*. For a section that does not meet the minimum transverse reinforcement requirements, LRFD Table 5.8.3.4.2-2 should be used to determine β and θ.

This process may be considered an improvement in accounting for the interaction between shear and flexure and attempting to control cracking at strength limit state.

For a non-prestressed concrete section not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified in LRFD Article 5.8.2.5, or having an overall depth of less than 400 mm, a value of 2.0 may be taken for β and a value of 45° may be taken for θ.

Transverse shear reinforcement should be provided if

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

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

$$A_v = 0.083\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(LRFD Eq. 5.8.2.5-1)}$$

If the reaction introduces compression into the end of the member, the critical section for shear is taken as the larger of $0.5d_v\cot\theta$ or d_v, measured from the face of the support (see LRFD Article 5.8.3.2 of the *Specifications*).

Torsion is most often not a major consideration. 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 the following:

1. cantilever brackets connected perpendicular to a concrete beam, especially if a diaphragm is not located opposite the bracket; and

2. concrete diaphragms used to make precast beams continuous for live load where the beams are spaced differently in adjacent spans.

62-1.05 Strut-and-Tie Model

Reference: Article 5.6.3

This method of modeling reinforced concrete components originated around 1900, but it has only recently been incorporated into the design code. Members, when loaded, indicate the presence of definite stress fields which can individually be represented by tensile or compressive resultant forces as their vectoral sums. It has been observed that the “load paths” taken by these resultants form a truss-like pattern which is optimum for the given loading and that the resultants are in reasonable equilibrium, especially after cracking. The compressive concrete paths are the struts, and the reinforcing steel groups are the ties. The model is shearless.

The model has significant application for bridge components and parts such as pier caps, beam ends, post-tensioning anchorage zones, etc. A thorough presentation of the model can be found in Chapter 8 of the *PCI Precast Prestressed Concrete Bridge Design Manual*, by Richard M. Baker and Jay A. Puckett, *Design of Highway Bridges based on AASHTO LRFD Design Specifications*, and in *Towards a Consistent Design of Structural Concrete*, by Schlaich, J., PCI Journal, Vol. 32, No. 3, 1987. The *LRFD Specifications* provides adequately for design; even if the model is not used for actual proportioning, it gives a fast check to ensure that no loose ends remain in design, especially for the appropriate anchorage of the steel.

Application of the model for a hammerhead pier is demonstrated in Figure 62-1B. There are five beams supported by the pier, of which two affect the design of a cantilever. There are several acceptable truss geometries. The one selected here assures that the struts, being parallel, are independent from each other. The scheme is indicative of the significance of a well-proportioned haunch. This design will yield approximately the same amount of steel in both ties. The steel in both ties is extended to the boundaries of their respective struts, then hooked down. The 90° hook of Tie #1 is further secured to the concrete by secondary steel, and the hook of Tie #2 is positioned in, and normal to, Strut #1.

This example was selected because of potential excessive cracking of pier heads invariably designed as beams. It is believed that normal beam design is unconservative for this application, due to the following:

1. discontinuity of steel too early;

2. an erroneous estimate for the location of maximum moment (usually taken at the face of the pier-column); and

3. anchoring the steel in cracked zones.

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

The model can also be used for the approximate analysis of the beam end. Figure 62-1C(a) shows a convenient way of checking the adequacy of reinforcement in the end-zone and the magnitude of compressive stresses in the web. In lieu of refined calculations, the angle θ may be assumed as 30°.

Figure 62-1C(b) indicates an application of the model to estimate the transverse forces in the bearing area to be resisted by the cage.

62-1.06 Fatigue

Reference: Articles 3.4.1, 3.6.1.4, and 5.5.3

The fatigue limit state is not normally a critical issue. Fatigue need not be considered for the deck where the permanent stress f_{min} is compressive and exceeds twice the maximum tensile live load stress.

Assuming r/h = 0.3, LRFD Equation 5.5.3.2-1 may be rearranged for easier interpretations as follows:

$$f_f + 0.33\, f_{min} \leq 161.5 \text{ MPa}$$

The *LRFD Specifications* presents a major change in computing f_f. It is the stress range due to 75% of a single truck per bridge (lane load excluded) with reduced impact (15%) and with the major axles of the truck at a constant spacing of 9 m, instead of all contributing lanes being loaded. Also, the *LRFD Specifications* specifies that, if the bridge is analyzed by the approximate distribution method, live load distribution factors for one design lane loaded shall be used.

62-1.07 Crack Control

Reference: Article 5.7.3.4

All concrete components shall be proportioned to control cracking. This design shall be in accordance with LRFD Article 5.7.3.4, except for an empirical deck design, which shall be in accordance with LRFD Article 9.7.2. When designing for crack-control, the following values for Z shall be used, unless a more severe condition is warranted.

1. 17 500 N/mm for footings;
2. 23 000 N/mm for deck and slab; and
3. 30 000 N/mm for all other components.

For a more detailed description of a slab design, see Section 61-2.02(05). Several bars, of minimum #13 size, at moderate spacing, are more effective in controlling cracking than one or two larger bars of equivalent area.

62-2.0 REINFORCING STEEL

62-2.01 Grades

Reference: Article 5.4.3.1

Steel reinforcing bars are manufactured as smooth or deformed bars. Deformed bars have ribbed projections that grip the concrete to provide a better bond between the steel and the concrete. Main bars, spirals, and ties are always deformed.

Reinforcing bars should be in accordance with ASTM A615M, Grade 420, with a yield strength of 420 MPa. The modulus of elasticity, E_s, should be taken as 200 000 MPa.

62-2.02 Sizes

Reinforcing bars are referred to by number, and they vary in size from #10 to #57. Figures 62-2A and 62-2B show the sizes, bar spacings, and various properties of the types of bars used.

To avoid handling damage, the minimum bar size shall be #13. A minor exception is for longitudinal ties in compression members, where #10 bars are allowable (see Section 67-3.03).

62-2.03 Concrete Cover

Reference: Article 5.12.3

See Figure 62-2C for criteria for minimum concrete cover for various applications. The values in Figure 62-2C are based on $0.40 \leq w/c \leq 0.50$. All clearances to reinforcing steel shall be shown on the plans.

62-2.04 Spacing of Reinforcement

Reference: Article 5.10.3

For minimum spacing of bars, see Figure 62-2D.

Fit and clearance of reinforcement shall be checked by means of calculations and large-scale drawings. Skews will tend to aggravate problems of reinforcing fit. Tolerances normally allowed for cutting, bending, and locating reinforcement shall be considered.

Some of the common areas of interference are as follows:

1. between slab reinforcement and reinforcement in monolithic end bents or intermediate bents;
2. vertical column bars projecting through main reinforcing in pier caps;
3. the areas near expansion devices;
4. anchor plates for steel girders;
5. at anchorages for a post-tensioned system; and
6. between prestressing (pretensioned or post-tensioned) steel and reinforcing steel stirrups, ties, etc.

The distance from the face of concrete to the center of the first bar should be shown for up to approximately a 2-m width. Where the distance between the first and last bars is such that the number of bars required results in spacings in increments of other than 5 mm, the bars should be shown to be equally spaced. For a greater width, one odd spacing should be used with increments of a 5-mm spacing for the rest.

62-2.05 Fabrication Lengths

See Figure 62-2A for maximum and normal bar lengths for fabrication. For ease of hauling and handling, the maximum length should be reduced where the location of the splice is arbitrary. The maximum length of bars extending above a horizontal joint (e.g., from a footing into a wall) should be 3 m.

62-2.06 Development of Reinforcement

Reference: Article 5.11.2

62-2.06(01) Development Length in Tension

Development length (l_d) or anchorage of reinforcement is required on both sides of a point of maximum stress at any section of a member.

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, epoxy coating 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 300 mm.

Figures 62-2E through 62-2H show the tension development length for both uncoated and epoxy coated bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class A concrete (f'_c = 24 MPa), use the development lengths shown for f'_c = 21 MPa unless calculated independently.

Development lengths shown in the figures for both uncoated and epoxy-coated bars must be multiplied by a factor of 2.0 for bars with a cover of d_b (bar diameter) or less, or with a clear spacing between bars of $2d_b$ or less. Development lengths shown for epoxy-coated bars may be multiplied by a factor of 0.80, if the cover is $3d_b$ or more and the clear spacing between bars is $6d_b$ or more.

62-2.06(02) Development Length in Compression

The standard procedure is to use tension development lengths for bars in either tension or in compression. This ensures that an adequate development length will be provided in a compression member that may be primarily controlled by bending.

62-2.06(03) Standard End Hook Development Length in Tension

Standard end hooks, utilizing 90° and 180° end hooks, are used 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. The values shown in Figures 62-2 I and 62-2L show the tension development lengths for both uncoated and epoxy-coated hooked bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class A concrete (f'_c = 24 MPa), use development lengths shown for f'_c = 21 MPa unless calculated independently.

See the figure in the commentary of Article C5.11.2.4.1 in the *LRFD Specifications* for hooked-bar details for the development of standard hooks.

<u>62-2.07 Splices</u>

Reference: Article 5.11.5

62-2.07(01) General

Two methods may be used to splice reinforcing bars: lap splices and mechanical splices. Lap splicing of reinforcing bars is the most common method. The plans should clearly show the locations and lengths of all lap splices. Due to splice lengths required, lap splices are not permitted for bars larger than #36. However, if bars larger than #36 are necessary, mechanical bar splices shall be used. Mechanical bar splices should also be considered in lieu of lap splices in highly congested areas. Mechanical splices are required for tension tie members.

No lap splices, for either tension or compression bars, should be less than 300 mm. See Section 703 of the *INDOT Standard Specifications* for additional splice requirements.

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

62-2.07(02) Lap Splices in Tension

Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar development length (l_d). There are three classes of tension lap splices: Class A, B, and C. Bars should be spliced at points of minimum stress.

For tension splices, the length of a lap splice between bars of different sizes shall be governed by the smaller bar.

Figures 62-2M through 62-2X show tension lap splices for both uncoated and epoxy coated bars for normal weight concrete with specified strengths of 21 and 28 MPa. For class A Concrete (f'_c = 24 MPa), use splice lengths shown for f'_c = 21 MPa unless calculated independently.

Splice lengths for spacing ≥ 150 mm, shown in the Figures for both uncoated and epoxy coated bars, must be multiplied by a factor of 2.0 for bars with a cover of d_b or less, or with a clear spacing between bars of $2d_b$ or less, where d_b equals the bar diameter. Splice lengths shown for epoxy-coated bars may also be multiplied by a factor of 0.8 if cover is $3d_b$ or more and clear spacing between bars is $6d_b$ or more.

62-2.07(03) Lap Splices in Compression

Lap splices in compression members are sized for tension lap splices. The design of compression members, such as columns, pier walls, and abutment walls, involves the combination of vertical and lateral loads. Therefore, the policy of requiring a tension lap splice accounts for the possibility that the member design may be primarily controlled by bending. Also, the increase in cost of additional splice reinforcement material is small.

62-2.07(04) Mechanical Splices

A second method of splicing is by means of mechanical splices, which are proprietary splicing mechanisms. The requirements for mechanical splices are found in Articles 5.11.5.2.2, 5.11.5.3.2, and 5.11.5.5.2 of the *LRFD Specifications*. All mechanical connectors shall develop not less than 125% of the specified yield strength of the bar regardless of the stress level in the bar.

62-2.07(05) Welded Splices

Splicing of reinforcing bars by means of welding is not permitted.

62-2.08 Hooks and Bends

Reference: Article 5.11.2.4

For standard hooks and bend diameters, see Figure 62-2Y. The values of A should be used for standard 90° hooks for both longitudinal reinforcement (end hooks) and transverse reinforcement (stirrups and tie hooks). For transverse reinforcement where the bar size is #10 or #13 and shorter tail lengths are required for better constructability, special hooks may be used. Dimensions and bend diameters of special hooks should be shown on the plans and should be in accordance with the CRSI *Manual of Standard Practice*. The total length of each bent bar should be rounded up to the next 20 mm. The legs of the bar should add up to this total. The difference must be added to a leg of the bar.

62-2.09 Epoxy-Coated Reinforcement

Reference: Articles 2.5.2.1.1 and 5.12.4

Epoxy-coated reinforcement should be used at the locations as follows:

1. the bridge deck;
2. the top 300 mm of a reinforced concrete slab bridge;
3. the end bents and wingwalls of an integral end bent beam and deck-type structure;
4. the end bents and wingwalls of a beam and deck-type structure where deck expansion joints are located at the ends of the structure;
5. above the footings of all interior substructure units that are located below deck expansion joints. For tall piers and bents, engineering judgment should be exercised;
6. concrete bridge railings;
7. bars extending into the deck from beams or substructures; and
8. reinforced concrete bridge approaches.

For all other locations, use uncoated bars. These include the following:

1. piers, bents, or abutments that are located adjacent to a pavement surface; and
2. a reinforced concrete retaining wall.

62-2.10 Bar Detailing

62-2.10(01) Standard Practice

The following presents standard practices for detailing reinforcing bars.

1. Reinforcing bars shall be called out in plan, elevation, and sections to clearly indicate the size, location, and spacing of the individual bars. The number of reinforcing bars shall be called out in only one view, usually the plan or elevation view. In other views, only the bar size and length (optional) or bar mark shall be called out.

2. Usually, in plan or elevation views, only the first bar and the last bar of a series of bars need be drawn, and the number of bars indicated between. In section views, all bars should be shown.

3. All dimensions on details are measured on centerlines of bars, except where cover, e.g., 50 cl., is indicated.

4. Straight bars will be designated by size and length (e.g., #13 x 4600).

5. Straight-bar lengths should be in 100-mm multiples, except for short vertical bars in railings and parallel wings, which should be in 25-mm multiples.

6. Bent bars are given a bar mark of which the first two numbers indicate the size of the bar and the last two numbers, 01 to 99, indicate the mark. Each bar mark is given a lower case letter suffix to indicate the location of the bar in the proper element of the structure (e.g., 2501a, 2502a). The following letters may be used as suffixes:

    a, b, c, d, f, h, k, m, n, p, r, s, t, u, v, w, x, y, and z.

    Assign letters in sequence with superstructure first and substructure last. For the substructure, assign letters in sequence for each abutment or bent except where these are detailed in pairs in which case the one letter is to apply to both.

7. Epoxy-coated bars will be suffixed by the letter E (e.g., #19E x 4600, 2501aE). If all bars are epoxy-coated, a note will suffice.

The following should be considered when selecting and detailing reinforcing steel.

1. Where possible, make similar bars alike to result in as few different bars in a structural element as practical.

2. When rounding off lengths of bars, one length should not encroach upon the minimum clearances.

3. Consideration shall be given to ease of placement of bars. A bar should not have to be threaded through a maze of other bars. The bars should be located so that they can be easily supported or tied to other reinforcement.

4. It may be more practical to lap two bent bars than to have a bar with five or six bends.

62-2.10(02) Bars in Section

Figure 62-2Z presents a section through a hypothetical member showing some accepted methods for detailing reinforcing steel. The following list describes some of the concerns and observations that should be considered when detailing reinforcing steel.

Sections shall be drawn to a large enough scale to clearly show reinforcing details.

1. Stirrups and other bars not shown end-on shall be drawn as single broken or unbroken lines for a scale less than 1:10, and as double unbroken lines for a scale of 1:10 or larger.

2. Bends of standard hooks and stirrups generally need not be dimensioned; however, all bends shall be drawn to scale.

3. Bars shown end-on shall be shown as small circles. The circles may be left open or may be shown as a dot. However, the symbol used shall be consistently applied on the drawing. If bars and holes will be shown, the bars shall be shown as solid.

4. Arrowheads pointing to the bar or circles drawn around the bar shall be the acceptable methods of detailing for bars shown end-on. Arrowheads shall point directly to the bar.

5. Sections cut at specific locations along a member are often preferred over a typical section for complex reinforcing patterns.

6. Corner bars enclosed by stirrups or ties should be shown at the corner of the bend (see Figure 62-2Z).

62-2.11 Bending Diagrams

The following presents standard practices when detailing bending diagrams.

1. All dimensions on bending diagrams are measured out-to-out of bars.
2. All bent bar partial dimensions shall be given to the nearest 5 mm.
3. The overall length of bent bars shall be rounded up to the next 20 mm.

See Figure 62-2AA for information on bending diagrams.

62-2.12 Cutting Diagrams

Two methods of showing cutting diagrams are provided. Other methods may be used at the discretion of the designer. The first is used where two sets of the same size bars are required and the second is used where only one bar of each size is required. Cutting diagrams are given a bar mark like bent bars. The first method is shown in an example of a skewed deck with the same bars in the top and bottom mat. Figure 62-2BB applies to the transverse steel in a bridge deck. The pertinent information should be determined as follows:

1. Determine the longest (B) and shortest (A) bars required to the nearest 20 mm.
2. Determine the number of bars required.
3. Divide the number of spaces (the number of bars minus 1) by the difference in length between the longest and shortest bars to obtain the increment. Round the increment to the nearest millimeter.
4. The length L is the sum of A + B.

The second method would be used such as in an asymmetric widening of a hammerhead pier. An even number of bars will be provided by this cutting group. Figure 62-2CC shows the cantilevered portion of a hammerhead pier.

1. Determine the longest (A) and shortest (D) bars required to the nearest 20 mm.
2. Determine the number of bars required.
3. Divide the number of spaces (number of bars minus 1) by the difference in length between the longest and shortest bars to obtain the increment N. Round the increment to the nearest millimeter.
4. Determine dimensions B and C as follows:

$$B \text{ or } C = \frac{A + D}{2 \pm 0.5N}$$

5. The length L = A + D = B + C. Adjust dimensions as necessary to make them fit this equation.

62-2.13 Bill of Materials

The following applies to the Bill of Materials.

1. Largest bar sizes shall be listed first.
2. For each bar size, bent bars shall be listed sequentially by number first followed by straight bars.
3. Straight bars shall be listed by decreasing length.
4. Subtotals of the mass should be provided for each bar size.
5. Plain and epoxy-coated bars shall be billed separately with totals for each.
6. There shall be separate Bills of Materials shown on the appropriate plan sheets for each structural element.
7. If two structural elements are very similar in dimension and reinforcement, it is permissible to combine the quantities into one Bill of Materials.

Figure 62-2DD illustrates a typical Bill of Materials for a reinforced concrete bridge approach.

62-3.0 REINFORCED CAST-IN-PLACE CONCRETE SLAB SUPERSTRUCTURE

62-3.01 General

The reinforced cast-in-place concrete slab superstructure is frequently used due to its suitability for short spans and its ease of construction. It is the simplest among all superstructure systems.

Section 62-3.0 presents information for the design of a reinforced cast-in-place concrete slab superstructure that amplifies or clarifies the provisions in the *LRFD Bridge Design Specifications*.

62-3.01(01) Materials

Reference: Article 5.4

Class C concrete should be used. See Figure 62-1A for concrete properties.

62-3.01(02) Cover

Reference: Article 5.12.3

Figure 62-3A presents criteria for minimum concrete cover for various structure elements. All clearances to reinforcing steel shall be shown on the detailed plans.

62-3.01(03) Haunches

Straight haunches are preferred to parabolic haunches because straight haunches are relatively easy to form yet result in relatively good stress flow.

In general, haunching is used to decrease maximum positive moments in a continuous structure by attracting more negative moments to the haunches and to provide adequate resistance at the haunches for the increased negative moments. It is a simple, effective, and economical way to enhance the resistance of a thin concrete slab. As illustrated in Figure 62-3B, there are three ways of forming the haunch. The parabolic shape (a) is the most natural in terms of stress flow, and certainly the most aesthetic, and it should be preferred for locations where the elevation is frequently in view. The parabolic haunch, however, is not the easiest to form and, as alternatives, the straight haunch (b), and the drop panel (c), should be considered where appropriate. The narrow pile cap (d), used in conjunction with extended pile substructures, does not qualify as an effective haunch.

Figure 62-3C depicts the elevation and plan of a three-span, continuous haunched slab bridge with an extensive skew. The preferable ratio between interior and end span is approximately 1.25 to 1.33 for economy, but the system permits considerable freedom in selecting span ratios. The ratio between the depths at the centerlines of interior piers and at the point of maximum positive moment should be between 2.0 and 2.5. Except for aesthetic reasons, the length of the haunch need not exceed the kL values indicated in Figure 62-3B, where L is the end span length. Longer haunches may be unnecessarily expensive and/or structurally counterproductive.

62-3.01(04) Substructures

The following describes typical practices for types of substructures used.

1. End Supports. Where possible, use integral end bents. In general, their use is not restricted by highway alignment nor skew. The maximum bridge length is 60 m for the use of integral end bents without a special analysis. See Section 59-2.02 for more information on end supports, including the use of non-integral end bents and abutments and the use of integral end bents where the bridge length exceeds 60 m.

2. Interior Supports. See Section 59-2.03 for typical practices for the selection of the type of interior support (e.g., piers, frame bents).

62-3.01(05) Minimum Reinforcement

Reference: 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 Articles 5.7.3.3.2 and 5.10.8 of the *LRFD Specifications*. The first is based on the cracking flexural strength of a component, and the second reflects requirements for shrinkage and temperature. In a slab superstructure, the two articles provide nearly identical amounts of minimum reinforcement in the majority of cases.

According to Article 5.14.4.1 of the *LRFD Specifications*, bottom transverse reinforcement, with the minimum provisions described above as notwithstanding, may be determined either by two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with 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 a heavily skewed and/or curved bridge, the analytical approach is recommended.

Section 62-3.05 gives a simplified approach for shrinkage and temperature steel requirements.

62-3.02 Computation of Slab Dead-Load Deflections

Reference: Article 5.7.3.6.2

For a concrete-deck-on-girder-type superstructure, the screed elevations should be provided in accordance with Section 61-4.02(01). For a simple span or a continuous span reinforced concrete slab superstructure, a dead-load deflection diagram showing the quarter-point deflections is provided on the plans. The contractor uses this information to develop screed elevations that will enable the contractor to pour the concrete slab at the proper final elevations. If a concrete slab superstructure is located within a superelevation transition, or if other geometric complications are present, screed elevations are provided at 1.5-m intervals.

The following criteria should be used in developing a dead-load deflection diagram.

1. Compute dead-load deflections due to the weight of the concrete slab at the span quarter points or at a closer spacing if more accuracy is desired.
2. Compute instantaneous deflections by the usual methods using formulas for elastic deflections.
3. For determining deflections, use the gross moment of inertia and modulus of elasticity shown in Figure 62-1A.
4. Round off values for deflections to the nearest 1 mm.

5. The deflection of the concrete slab caused by the weight of a concrete barrier railing is insignificant and may be ignored when developing the slab dead-load deflection diagram.
6. Do not include the effects of form settlement or crushing. This is the contractor's responsibility.

62-3.03 Construction Joints

Transverse construction joints are not permitted. The INDOT *Standard Specifications* provide construction requirements where transverse construction joints are unavoidable because concreting is interrupted due to rain or other unavoidable events.

Longitudinal construction joints are also undesirable. However, 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 slab must be poured in one or more pours. An optional longitudinal keyway construction joint should be shown on the plans at the centerline of roadway. The contractor may request permission to eliminate the construction joint by providing information specific to the proposed method of placing concrete and equipment to be used.

Where phased construction is not anticipated, transverse reinforcing steel may be lapped at the optional longitudinal construction joint. If the structure will be built in phases, show the entire lap splice for all transverse reinforcing steel on the side of the construction joint that will be poured last.

62-3.04 Longitudinal Edge Beam Design

Reference: Articles 5.14.4.1, 9.7.1.4, and 4.6.2.1.4

An edge beam must be provided along the each slab edge. Structurally continuous barriers may only be considered effective for the service limit states, not the strength or extreme-event limit states. An edge beam can be a thickened section and/or a more heavily reinforced section composite with the slab. The width of the edge beam may be taken to be the width of the equivalent strip as specified in Article 4.6.2.1.4b.

62-3.05 Shrinkage and Temperature Reinforcement

Reference: Articles 5.6.2 and 5.10.8

Evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep, and movements of supports is not necessary.

Shrinkage and temperature reinforcement as a function of slab thickness is given in Figure 62-3D.

62-3.06 Reinforcing Steel and Constructibility

The following practices for reinforcing steel should be met to improve the constructability.

1. The maximum reinforcing bar size shall be #36.
2. The minimum spacing of reinforcing bars shall preferably be 150 mm.
3. Longitudinal steel should be detailed in a 2-bar alternating pattern, with one of the bars continuous through the slab. The maximum difference between the bars shall be two standard bar sizes.

Vertical steel, other than that required to keep the longitudinal negative moment reinforcement floating, may not be required. Article 5.11.1.2 of the *LRFD Specifications* presents specifications for the portion of the longitudinal positive moment reinforcement that must be extended to the next support point in excess of that required by the factored maximum moment diagram. Similarly, there is a more stringent provision addressing the location of the anchorage for the longitudinal negative moment reinforcement.

62-3.07 Drainage Outlets

Reference: Article 2.6.6

Chapter Thirty-three discusses the hydrological and hydraulic analyses for a bridge deck. The following specifically applies.

1. Type of Inlet. Use the deck drains shown on the INDOT *Standard Drawings*. The deck drains are designed for reinforced concrete slab bridges only. The drain is a 150-mm PVC pipe set into the deck. The small deck drains have limited hydraulic capacity; therefore, the standard spacing is approximately 1800 mm. A 15-mm depression, which extends 300 mm transversely from the face of the curb, slightly increases the capacity. The PVC pipe must clear the bent cap faces by 600 mm.

2. Concrete Barrier Rails (Scuppers). Scuppers through concrete barrier railings are permitted only on a local public agency structure.

62-3.08 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.

62-3.09 Distribution of Live Load

Reference: Article 4.6.2.3

Section 60-3.02 discusses the application of vehicular live load, and Section 61-2.03 discusses the longitudinal application of the Strip Method. The following specifically applies to the distribution of live loads.

1. For a continuous superstructure 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. Using Equation 4.6.2.3-3 from the *LRFD Specifications* for the reduction of moments in a skewed structure will not usually significantly change the reinforcing steel requirements. Therefore, for simplicity of design, use of the reduction factor *r* is not required.

62-3.10 Shear Resistance

Reference: Article 5.14.4.1

The moment design in accordance with Article 4.6.2.3 of the *LRFD Specifications* may be considered satisfactory for shear.

62-3.11 Minimum Thickness of Slab

Reference: Article 2.5.2.6.3

The minimum slab thickness should be in accordance with Table 2.5.2.6.3-1 of the *LRFD Specifications*. In using the equations in the LRFD Table, the assumptions are as follows:

1. S is the length of the longest span.

2. The calculated thickness includes the 15-mm sacrificial wearing surface.
3. The thickness used may be greater than the value obtained from the Table.
4. The thickness used may be less than the value obtained from the Table as long as the live load deflection does not exceed the criteria shown in LRFD Article 2.5.2.6.2.

62-3.12 Development of Flexural Reinforcement

Reference: Article 5.11.1.2

Article 5.11.1.2 of the *LRFD Specifications* 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.

62-3.13 Skewed Reinforced Concrete Slab Bridge

For a skew angle of less than 45°, the transverse reinforcement is permitted to run parallel to the skew, providing for equal bar lengths. For a skew angle of 45° or greater, the transverse reinforcement should be placed perpendicular to the longitudinal reinforcement. This provision concerns the direction of principal tensile stresses as they develop in a heavily skewed structure and is intended to prevent excessive cracking.

Special slab superstructure designs or modifications to the integral end supports are not required for a heavily skewed and/or curved structure. The provisions are based upon performance of relatively small span structures constructed to date. These slab superstructures have included skews in excess of 50° and moderate curvatures. Any significant deviation from successful past practice should be reviewed. See Figure 62-3C.

62-3.14 Design Details for Integral Caps at Slab Superstructure

The following presents specific design details which represent typical practices for the design of integral caps.

1. The standard pile cap dimensions are 750 mm in width and a depth of 450 mm plus the slab thickness.
2. For a skewed structure, the 750-mm width dimension is measured perpendicular to the skew.
3. All transverse reinforcement (stirrups) in the caps is placed perpendicular to the skew.
4. Minimum concrete cover for cap reinforcing steel is 50 mm.

5. Standard pile embedment into the end-bent and interior-bent caps is 300 mm.
6. Support bars and coping stirrup bars are used to provide support for the top steel in the slab. Stirrup bars shall be placed parallel to the skew.
7. A 20-mm, half-round drip bead shall be located under the deck, 150 mm in from the face of coping.

The following applies to the level or sloped profile for the bottom of a bent cap.

1. Either a level or sloped profile of the bottom of a bent cap can be easily formed.

2. The profile of the bottom of the bent cap can be made level if the different in top-of-slab elevations at the left and right copings, along the centerline of the bent cap, is 75 mm or less. For a difference greater than 75 mm, slope the bottom of the cap from coping to coping.

Figures 62-3E through 62-3H present the typical practices for slab-superstructure cap detailing.

62-3.15 Transverse Shrinkage and Temperature Reinforcement in the Top of the Slab at the Bent Caps

Reference: Article 5.10.8.1

Article 5.10.8.1 states that reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes. Top longitudinal cap flexural reinforcement cannot be considered effective reinforcement for transverse shrinkage and temperature stresses if this steel is located significantly below the surface of the concrete slab.

62-3.16 Fatigue Limit State

Reference: Article 5.5.3

The fatigue limit state does not generally control the area of steel required at the points of maximum moment. However, it may control at bar cut-off points. The stress range, f_f, must satisfy LRFD Equation 5.5.3.2-1:

$$f_f \leq 145 - 0.33 f_{min} + 55(r/h)$$

The section properties shall be based on cracked sections where the sum of the stresses due to unfactored permanent loads plus 1.5 times the fatigue load is tensile and exceeds $0.25\sqrt{f'_c}$. Sections in stress reversal areas should be analyzed as doubly reinforced sections.