SCDOT Bridge Design Manual

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FOREWORD

The SCDOT Bridge Design Manual has been developed to provide engineers with the Department’s standard bridge design policies and practices. Engineers should attempt to meet all of the criteria presented in the Manual, while fulfilling the Department’s mission of providing a safe and efficient transportation system for the State. However, the criteria presented in the Manual should not be considered as a standard that must be met in all circumstances. Engineers must consider economic impacts, aesthetics, and the social and cultural resources of the project area and request modifications to the criteria when conditions warrant. Because it is impossible to address every issue that bridge engineers will encounter, sound engineering judgment must be exercised when conditions arise that are not specifically covered in the Manual.

The SCDOT Bridge Design Manual was prepared based on the 3rd Edition of the AASHTO LRFD Bridge Design Specifications, current through the 2005 Interims.

ACKNOWLEDGEMENTS

The SCDOT Bridge Design Manual was developed by the Bridge Design Section with assistance from the consulting firm of Roy Jorgensen Associates, Inc., Professor Dennis Mertz of the University of Delaware, and the consulting firm The LPA Group, Inc. The South Carolina Federal Highway Administration Division Bridge Engineer provided input and oversight during the development process.

REVISION PROCESS

The SCDOT Bridge Design Manual is intended to provide current bridge design policies and procedures for use in developing State highway projects. To ensure that the Manual remains up-to-date and appropriately reflects changes in SCDOT’s needs and requirements, its contents will be updated on an ongoing basis. It is the responsibility of the Manual holder to keep the Manual updated.

The SCDOT Bridge Design Section will be responsible for evaluating changes in bridge design literature (e.g., updates to the LRFD Specifications, the issuance of new research publications, revisions to Federal regulations) and will ensure that those changes are appropriately addressed through the issuance of revisions to the Manual. It is important that users of the Manual inform SCDOT of any inconsistencies, errors, need for clarification, or new ideas to support the goal of providing the best and most up-to-date information practical. Comments may be forwarded to the State Bridge Design Engineer.

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CHAPTER 17
BRIDGE DECKS

Sections 3, 4, and 9 of the LRFD Bridge Design Specifications present the AASHTO criteria for the structural design of bridge decks. Section 3 specifies loads for bridge decks, Section 4 specifies their analyses, and Section 9 specifies the resistance of bridge decks. Unless noted otherwise in this Chapter of the SCDOT Bridge Design Manual, the LRFD Specifications applies to the design of bridge decks in South Carolina. This Chapter presents information on specific SCDOT practices for bridge decks.

17.1 BACKGROUND

17.1.1 Bridge Decks and Superstructures

The LRFD Specifications encourages the integration of the deck with the primary components of the superstructure by either composite or monolithic action. In some cases, the deck alone is the superstructure. The LRFD Specifications refers to this as a “slab superstructure”; SCDOT refers to these as “flat slabs.” More commonly, the deck in conjunction with its supporting components comprises the superstructure.

This Chapter documents SCDOT criteria on the design of bridge decks that are constructed compositely in conjunction with concrete I-beams or steel I-girders. Chapter 15 discusses the design of flat slabs.

17.1.2 Durability of Concrete Bridge Decks

Reference: LRFD Articles 1.2, 2.5.2.1.1, and 5.12

As stated in the commentary to LRFD Article 2.5.2.1.1, the single most prevalent bridge maintenance problem is the deterioration of concrete bridge decks. LRFD Article 5.12 discusses measures to enhance the durability of concrete components.

The distress of bridge decks, and their premature replacement, has become a serious problem in the United States. In Article 1.2, the LRFD Specifications defines the design life of new bridges as 75 years. Thus, designers are compelled to re-evaluate conventional wisdom regarding the long-term performance of concrete bridge decks.

17.1.3 Protection of Reinforcing Bars

See Section 15.3 for the SCDOT corrosion protection policy for reinforcing bars.

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17.2 "STRIP METHOD"

17.2.1 Application of the "Strip Method" to Composite Concrete Decks

Reference: Appendix to LRFD Section 4

The application of the strip method to composite concrete decks is represented by a design table for deck slabs in the Appendix to Section 4 of the LRFD Specifications (LRFD Table A4-1). An introduction to the LRFD Table discusses its application.

LRFD Table A4-1 shall be used to design the concrete deck reinforcement. LRFD Table A4-1 tabulates the resultant live-load moments per unit width for slab steel design as a function of the girder spacing, S. Negative moments are distinguished from positive moments and are tabulated for various design sections as a function of the distance from the girder centerline to the design section. LRFD Article 4.6.2.1.6 specifies the design sections to be used.

17.2.2 Empirical Deck Design

SCDOT prohibits the use of the empirical deck design.
17.3 DESIGN DETAILS FOR BRIDGE DECKS

17.3.1 General

The following general criteria applies to bridge deck design:

1. **Thickness.** The depth of reinforced concrete decks shall not be less than 8 in.

2. **Reinforcement Steel Strength.** The specified yield strength of reinforcing steel shall be 60 ksi.

3. **Reinforcement Cover.** Typically, the bottom reinforcement cover shall be a minimum of 1 in. The top reinforcement cover shall be a minimum of 2½ in, which includes a ¼-in sacrificial wearing surface. The primary reinforcement shall be the closer reinforcement to the concrete face. See Figure 15.3-2 for additional concrete cover criteria.

4. **Reinforcing Bar Spacing.** A minimum of 1½ in (based on nominal bar diameters) vertically between the top and bottom reinforcing mats shall be maintained. Where conduits are present between mats, the 1½ in must be increased. A minimum horizontal spacing of 5½ in on center shall be maintained between adjacent bars within each mat. These minimum spacings are required to ensure the proper consolidation of the concrete between bars. The maximum horizontal reinforcing bar spacing is 9 in for primary (transverse) steel. See Section 15.3 for additional information on reinforcing bar spacing.

5. **Reinforcing Bar Size.** The minimum reinforcing bar size used for slab reinforcement is a #5 bar. However, #4 bars may be used in deck overhangs where they are bundled with the primary reinforcing. For designs that require a slab thickness exceeding 8½ in, the designer may elect to use #6 bars for the primary reinforcing.

6. **Sacrificial Wearing Surface.** The 2½ in top reinforcement concrete cover includes ¼ in that is considered sacrificial. Its weight shall be included as a dead load, but its structural contribution shall not be included in the structural design.

7. **Concrete Strength.** The specified 28-day compressive strength of concrete for bridge decks and approach slabs shall be 4.0 ksi.

8. **Length of Reinforcement Steel.** The maximum length of reinforcing steel in the deck shall be 40 ft for galvanized reinforcing bars and 60 ft for black (uncoated) reinforcing bars.

9. **Placement of Transverse Reinforcing Bars on Skewed Bridges.** The following applies:
   a. **Skews ≤ 30°:** Place the transverse reinforcing steel parallel to the skew.
   b. **Skews > 30°:** Place the transverse reinforcing steel perpendicular to the longitudinal reinforcement.
See Section 17.3.4 for a definition of skew angle and for structural considerations related to skewed reinforcing bar placement.

10. Splices/Connectors. Use lap splices for deck reinforcement unless special circumstances exist. Mechanical connectors may be used where clearance problems exist or on a phase construction project that precludes the use of lap splices. See Section 15.3 for more discussion on splices.


17.3.2 Dimensional Requirements for Concrete Decks

17.3.2.1 General

Although the build-down varies across the width of the flange and the length of the girder, in all cases there shall be a minimum of $\frac{1}{2}$-in build-down. Consider the camber tolerance when calculating the $\frac{1}{2}$-in minimum.

The Control Dimension “D” is measured at the centerline of bearing for all girders, and varies in the span to compensate for variations in camber, superelevation ordinate, and vertical curve ordinate if necessary.

The build-down for deck slabs should be detailed flush with the vertical edge of the top flange.

17.3.2.2 Build-Down Dimensions for Steel Girders

Figure 17.3-1 illustrates the controlling factors used to determine the build-down dimension for steel plate girders. Figure 17.3-2 illustrates a steel rolled beam.

17.3.2.3 Build-Down Dimensions for Concrete Beams

Figure 17.3-3 illustrates the controlling factors used to determine the build-down dimension for concrete beams.

17.3.3 Stay-in-Place Forms

Steel stay-in-place forms are allowed on all projects having beams or girders. Design loads for stay-in-place forms shall be applied for all girder/beam bridges and consist of 0.016 ksf for the metal forms applied over the areas of the forms. Field welding of the stay-in-place forms to steel flanges is prohibited. Stay-in-place forms are not allowed in bays having longitudinal joints. If the contractor elects not to use stay-in-place forms, the camber calculations must be modified accordingly because the assumed weight of the non-existent forms was originally included.
Control Dimension \( D = T + X \)

\( X = 3'' \) or
\( 1 \frac{1}{4}'' + \) the Thickest Top Flange,
Whichever is Greater

**BUILD-DOWN DIMENSION FOR STEEL PLATE GIRDERS**

*Figure 17.3-1*

**BUILD-DOWN DIMENSION FOR STEEL ROLLED BEAMS**

*Figure 17.3-2*
17.3.4 Skewed Decks

Reference: LRFD Article 9.7.1.3

Skew is defined by the angle between the end line of the deck and the normal drawn to the longitudinal centerline of the bridge at that point. See Figure 17.3-4. The two end skews can be different. In addition to skew, the behavior of the superstructure is also affected by the span-length-to-bridge-width ratio.

The LRFD Specifications generally implies that the effects of skew angles not exceeding 30° can be neglected for concrete decks, but the LRFD Specifications assumes the typical case of bridges with relatively large span-length-to-bridge-width ratios. Figure 17.3-4 illustrates four combinations of skew angles 30° and 50° and length-to-width ratios of 3:1 and 1:3.

Both the 50° skew and the 1:3 length-to-width ratio are considered extreme values for bridges, but this often occurs where the deck constitutes the top slab of a culvert. It can be judged visually that both combinations with 30° skew may be orthogonally modeled for design with the skew ignored.
The Commentary to Section 9 of the LRFD Specifications provides valid arguments supporting the limit of 30° concerning the direction of transverse reinforcement. It suggests that running the transverse reinforcement parallel to a skew larger than 30° will create a structurally undesirable situation in which the deck is essentially unreinforced in the direction of principal stresses. It is required that, for skew > 30°, the transverse reinforcement must be set perpendicular to the beams or longitudinal reinforcement.

The combination of 50° skew and \( L/W = 1:3 \), as indicated in Figure 17.3-4, produces an unusual layout. If the deck is a cast-in-place concrete slab without beams, the primary direction of structural action is one being perpendicular to the end line of the deck. Because of the geometry of the layout, consider running the primary reinforcement in that direction and fanning it as appropriate in the side zone. With this arrangement, the secondary reinforcement could then be run parallel to the skew, thus regaining the orthogonality of the reinforcement as appropriate for this layout.

17.3.5 Deck Pouring Sequence

Reference: LRFD Article 2.5.3

Maximum specified pouring rate: 60 \( \text{yd}^3/\text{hr} \) (300 \( \text{yd}^3 \) in 5 hours)
Minimum specified pouring rate: 45 \( \text{yd}^3/\text{hr} \) (225 \( \text{yd}^3 \) in 5 hours)
17.3.5.1 General

The need for a slab pouring sequence in the bridge plans will be based on the volume of concrete in the bridge deck as follows:

- Less than 225 yd$^3$ — not needed
- 225 yd$^3$ to 300 yd$^3$ — case-by-case decision
- Greater than 300 yd$^3$ — required

The 225-yd$^3$ and 300-yd$^3$ thresholds are calculated based on a minimum specified pouring rate of 45 yd$^3$/hr and a maximum specified pouring rate of 60 yd$^3$/hr, respectively, for five hours. If a pouring rate greater than 45 yd$^3$/hr is needed, the plans shall indicate the required pouring rate.

The bridge deck pouring sequence that is indicated in the contract documents is determined by the designer considering factors such as size of pour, configuration of the bridge, potential placement restrictions, direction of placement, deck tensile stresses, and any other special circumstances that might affect the bridge deck placement.

Where required, the bridge designer will present in the bridge plans the sequence of placing concrete in various sections (separated by transverse construction joints) of deck slabs on continuous spans. The designated sequence avoids or minimizes the dead-load tensile stresses in the slab during concrete setting to minimize cracking, and the sequence should be arranged to cause the least disturbance to the portions placed previously. In addition, for longer span steel girder bridges, the pouring sequence can lock-in stresses far different than those associated with the instantaneous placement typically assumed in design. Therefore, in these bridges, the designer shall consider the pouring sequence in the design of the girders.

Deck placement shall be uniform and continuous over the full width of the superstructure. The first pours shall include the positive-moment regions in all spans. The final pours shall include the negative-moment regions and shall not be placed until a minimum of 72 hours has elapsed from the start of the preceding pour. For pours on a grade of 3% or greater, the direction of pouring should be uphill.

Figure 17.3-5 illustrates a sample pour sequence diagram for continuous prestressed concrete I-beams made continuous for live load. The cast-in-place diaphragm over the bent is cast integrally at the same time as the deck above it. Also, see Chapter 6 for information on the presentation of the slab pouring sequence detail. For a continuous steel bridge, the pouring sequence will be similar, but the negative-moment regions are longer. The extent of the negative-moment regions is project-specific and shall be determined for each situation.

Prestressed concrete beams made continuous for live load and superimposed dead load shall be treated as a special case. The deck segment and diaphragm over the support provide continuity for live load in the superstructure after the previously poured center regions of the deck have been poured as simple span loads.
TYPICAL POUR DIAGRAM
(Continuous Prestressed Concrete I-Beam)

Figure 17.3-5

Note: The direction of pour should be shown for each pour.
For integral end bents, the end wall concrete shall be cast concurrently with the deck pour of the end span.

### 17.3.5.2 Transverse Construction Joints

Where used, transverse construction joints should be placed parallel to the transverse reinforcing steel.

Place a transverse construction joint in the end span of bridge decks on steel superstructures where uplift is a possibility during the deck pour. A bridge with an end span relatively short (60% or less) when compared to the adjacent interior span is most likely to produce this form of uplift. Uplift during the deck pour can also occur at the end supports of curved decks and in superstructures with severe skews. If an analysis shows that uplift might occur during a deck placement, require a construction joint in the end span and require placing a portion of the deck first to act as a counterweight.

### 17.3.6 Longitudinal Construction Joints

Longitudinal construction joints in bridge decks can create planes of weakness that frequently cause maintenance problems. In general, construction joints are discouraged, and their use should be minimized. The following will apply to longitudinal construction joints:

1. **Usage.** Construction joints need not be used on decks having a constant cross section where the width is less than or equal to 60 ft. For deck widths greater than 60 ft (i.e., where the screeding machine span width must exceed 60 ft), the designer shall make provisions to permit placing the deck in practical widths. For decks wider than 90 ft, the designer shall detail either a longitudinal open joint or a longitudinal closure pour, preferably not less than 3 ft in width. Lap splices in the transverse reinforcing steel shall be located within the longitudinal closure pour. Such a joint should remain open as long as the construction schedule permits to allow transverse shrinkage of the deck concrete. The designer should consider the deflections of the bridge on either side of the closure pour to ensure proper transverse fit up. See Section 21.1 for more information on longitudinal open joints.

2. **Location.** If a construction joint is necessary, do not locate it underneath a wheel line. Preferably, a construction joint should be located outside the beam flange.

3. **Closure Pours.** For staged construction projects where the deflection from the deck slab weight exceeds ½ in, a closure pour shall be used to connect the slab between stages. A closure pour serves two useful purposes: It defers final connection of the stages until after the deflection from the deck slab weight has occurred, and it provides the width needed to make a smooth transition between differences in final grades that result from design calculations or construction tolerances. Good engineering practice dictates that the closure width should relate to the amount of dead-load deflection that is expected to
occur after the closure is placed. A minimum closure width of 3 ft is recommended. When a closure pour is used, the following apply:

- Stay-in-place forms shall not be used under the closure pour.
- Diaphragms/cross frames in the staging bay of structural steel beams or girders shall not be rigidly connected until after the adjacent stages of the deck have been poured. Omit concrete diaphragms in the staging bay of prestressed concrete beams.
- Reinforcing steel between different stages shall not be tied or coupled until after the adjacent stages of the deck have been poured.

17.3.7 Bridge Deck Overhangs

Reference: LRFD Article 9.7.1.5

17.3.7.1 Width

Bridge deck overhang is defined as the distance between the centerline of the exterior girder or beam to the outside edge of the deck (i.e., behind the bridge rail). Typically, the projection of the bridge deck slab past the exterior beam or girder is constructed by bracing the falsework against the web or bottom flange of the exterior beam or girder. Large overhang widths will cause excessive lateral distortion of the bottom flange and web of the beam or girder. Section 12.2 provides more information on bridge overhang widths.

17.3.7.2 Overhang Treatments

Figure 17.3-6 shows typical overhang treatments for a steel girder bridge for a normal crown and for the low and high sides of superelevation. Details for concrete beam bridges are similar using the build-down as shown in Figure 17.3-7.

Figure 17.3-8 shows typical overhang treatments at expansion joints.

17.3.7.3 Design Details

The following details pertain to the edge of deck (see Figure 17.3-7):

1. **Chamfer.** Provide a ¼" chamfer at the top and bottom of the edge of the deck slab.
STEEL GIRDER OVERHANG TREATMENTS

Figure 17.3-6

Note: Concrete beam bridges are handled similarly with the build-down shown in Figure 17.3-7.

D = Control Dimension
W = Overhang Width
T = Slab Depth
2. **Slip Forming/Transverse Reinforcing Bar Ends.** Bridge deck slabs shall be designed to extend 1½ in past the back face of the barrier parapet to accommodate slip forming. The transverse reinforcing bar lengths in the deck slab should be computed to maintain 3 in of clearance from the edge of slab for construction tolerances.

3. **Drip Groove.** Locate the drip groove 2 in from the edge of the slab.
FOR SKEW ANGLES OF 10 DEGREES OR LESS

FOR SKEW ANGLES GREATER THAN 10 DEGREES

PART PLAN OF DECK CANTILEVER AT EXPANSION JOINT

Figure 17.3-8
17.4 APPROACH SLABS

17.4.1 Usage

See Section 12.2.

17.4.2 Design Criteria

See the SCDOT Bridge Drawings and Details, available at the SCDOT website, for the typical approach slab design. The roadway ends of approach slabs should be designed parallel to the bridge ends. The following design criteria applies to approach slabs:

1. Materials. Class 4000 concrete and Grade 60 reinforcing bars shall be used in the design of all approach slabs.

2. Length. Approach slabs shall be 20 ft long measured parallel to the centerline of roadway.

3. Thickness/Concrete Cover. The thickness of the approach slab shall be 12 in, with 2 in of concrete cover to the top reinforcing bars and 3 in of concrete cover below the bottom reinforcing bars.

4. Reinforcement. The following applies:
   - The top reinforcing steel that is parallel to the roadway shall be #7 bars at 12 in on center.
   - For flat slabs and cored slabs, the bottom reinforcing steel that is parallel to the roadway shall be #7 bars at 6 in on center.
   - For deck slabs on girder/beam bridges, the bottom reinforcing steel that is parallel to the roadway shall be #9 bars at 6 in on center.
   - The top and bottom distribution steel shall be #5 bars at 12 in on center.

5. Approach Slab Connections. All approach slabs shall be doweled to the end bent or pavement rest with #6 bars at 12 in on center. The anchors shall be detailed to act as a hinge so that the approach slab can rotate downward without stress as the embankment settles. The minimum pavement rest dimension is 8 in.

6. Approach Slabs and Grade Location. Where concrete pavement is used for the approaching roadway, approach slabs shall be constructed at grade. Where asphalt pavement is used for the approaching roadway, approach slabs shall be constructed 2 in below grade.
7. **Live Load.** The approach slab shall be modeled as a simple span for the determination of live-load reactions on the end bent. Where an approach slab is used, the live-load surcharge shall not be applied to the end bent.

8. **Dead Load.** SCDOT policy is to include one-half of the dead load of the approach slab as an end bent dead load.

### 17.4.3 Special Conditions

When any of the following special conditions exist, the designer shall evaluate the design criteria for approach slabs in Section 17.4.2 and redesign as needed:

1. **Skews.** Where skews of $30^\circ$ or greater exist, a redesign of the approach slab may be necessary.

2. **Deep End Spans.** The approach slab design should be reevaluated where the structure depth equals or exceeds one-half of the approach slab length.

3. **Sidewalks.** Where the project requires sidewalks on the bridge, the approach slab must be widened to allow for the sidewalks.

4. **Sleeper Slabs.** When sleeper slabs are used, the approach slab must be designed to span the entire distance between the sleeper slab and the pavement rest.