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CHAPTER SIXTY-TWO

REINFORCED CONCRETE

The *LRFD Bridge Design Specifications* Section 5 specifies the design requirements for concrete in all structural elements. This Chapter provides 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 Material Properties

Reference: Article 5.4

The minimum yield strength for reinforcing steel should be taken as 60 ksi.

Figure 62-1A provides 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 an under-reinforced rectangular section. Stresses in both top and bottom steel mats are taken at yield, while the concrete stress block is assumed to be rectangular with an intensity of $0.85 f_c'$ 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:

$$c = \frac{a}{\beta_1}$$

The factor β_1 should be taken as 0.85 for concrete strength not exceeding 4.0 ksi. For concrete strength exceeding 4.0 ksi, β_1 should be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi. However, β_1 should not be taken to be less than 0.65, in accordance with *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

LRFD Specifications Article 5.7.3.3.1 regulates the maximum allowable steel for reinforced and prestressed members by limiting the c/d_e ratio to 0.42. The *LRFD Specifications* prohibits 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 *LRFD* Equation C5.7.3.3.1-1 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 be certain 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 control.

Use *LRFD* 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 (kip-in.)
f_r = modulus of rupture of concrete as specified in *LRFD* Article 5.4.2.6 (ksi)
y_t = distance from the neutral axis to the extreme tension fiber (in.)

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,

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

* * * * * * * * * *

Example 62-1.1

For b = 12 in., h = 8 in. and f_c' = 4 ksi:

$$M_{cr} = (0.04)(12)(8^2)\sqrt{4} = 61.4k - in.,$$

d = 6.75 in., and f_y = 60 ksi

$$B = \frac{(-1.7)(bdf_c')}{f_y} = \frac{(-1.7)(12)(6.75)(4)}{60} = -9.18\ in^2$$

$$C = \frac{2.267M_{cr}bf_c'}{f_y^2} = \frac{(2.267)(61.4)(12)(4)}{60^2} = 1.86\ in^4$$

from which,

$$A_s = 0.5\left(-B - \sqrt{B^2 - 4C}\right) = 0.207\ in^2 \qquad \text{(Equation 62-1.1)}$$

or a ratio of $\rho = 0.207 \div (12 \times 8) = 0.0022$

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

* * * * * * * * * *

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 a typical bridge girder, slab, or other region 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 a region near a discontinuity, such as an abrupt change in cross-section, opening, coped (dapped) end, deep beam, or corbel, 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:

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

or

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

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

LRFD Equation 5.8.3.3-2 represents an upper limit of V_n to ensure 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:

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

The contribution of the web reinforcement is 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 where the web shear reinforcement is vertical (α = 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 including 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 a section that does not include 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 including at least the minimum amount of transverse reinforcement specified in *LRFD* Article 5.8.2.5, or having an overall depth of less than 16 in., a value of 2.0 may be taken for β and a value of 45 deg may be taken for θ.

Transverse shear reinforcement should be provided if the following applies.

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

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

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

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).

Torsion is most often not a major consideration. Where torsion effects are present, the member should be designed in accordance with *LRFD* Articles 5.8.2 and 5.8.3.6. A situation that may require a torsion design includes the following:

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

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

Members, if loaded, indicate the presence of definite stress fields which can individually be represented by tensile or compressive resultant forces as their vectoral sums. 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 the *PCI Precast Prestressed Concrete Bridge Design Manual*, Chapter 8, *Design of Highway Bridges based on AASHTO LRFD Design Specifications*, and in *Towards a Consistent Design of Structural Concrete*, PCI Journal, Vol. 32, No. 3, 1987. The *LRFD Specifications* provides adequately for design. If the model is not used for actual proportioning, it provides 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. The truss geometry selected here ensures 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-deg hook of Tie No. 1 is further secured to the concrete by secondary steel, and the hook of Tie No. 2 is positioned in, and normal to, Strut No. 1.

This example was selected because of the potential excessive cracking of a pier head invariably designed as a beam. Normal beam design is unconservative for this application, due to the following:

1. early discontinuity of steel;

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. Thus, the bonding capacity of cracked concrete cannot be considered completely reliable. Improperly-anchored steel is a design consideration in which mistakes are made, and the *LRFD Specifications* 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 deg.

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

References: 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:

$$f_f + 0.33\ f_{\min} \ \leq\ 23.4\ ksi$$

The *LRFD Specifications* shows a 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 30 ft, 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 should be used.

62-1.07 Crack Control

Reference: Article 5.7.3.4

Each concrete component should be proportioned to control cracking. The design should be in accordance with *LRFD* Article 5.7.3.4, except for an empirical deck design, which should be in accordance with *LRFD* Article 9.7.2. If designing for crack-control, the following values for Z should be used, unless a more severe condition is warranted.

1. 100 kip/in. for footing;
2. 130 kip/in. for deck and slab; and
3. 170 kip/in. for all other components.

For a more detailed description of a slab design, see Section 61-2.02(05). Several bars, of minimum #4 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 Grade

Reference: Article 5.4.3.1

Steel reinforcing bars are manufactured as either 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.

The yield strength of reinforcing bars should be taken as 60 ksi. The modulus of elasticity, E_s, should be taken as 29,000 ksi.

62-2.02 Sizes

Reinforcing bars are referred to by number, and they vary in size from #3 to #18. Figures 62-2A and 62-2B show the sizes, bar spacings, and properties of the types of bars used.

To avoid handling damage, the minimum bar size should be #4. Longitudinal ties in compression members may be #3 (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 should 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 should be considered.

Some of the common locations of interference are as follows:

1. between slab reinforcement and reinforcement in a monolithic end bent or intermediate bent;
2. vertical column bars projecting through main reinforcement in a pier cap;
3. the area near an expansion device;
4. anchor plates for steel girders;
5. at anchorages for a post-tensioned system; or
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 5-ft 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 0.25 in., the bars should be shown to be equally spaced. For a greater width, one odd spacing should be used with increments of a 0.25 in. 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 10 ft.

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 12 in.

Figures 62-2E through 62-2H show the tension development length for both uncoated and epoxy coated bars for normal weight concrete with specified 28-day strength of 3.0 ksi or 4.0 ksi. For Class A concrete (f'_c = 3.5 ksi), use the development lengths shown for f'_c = 3.0 ksi 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-deg or 180-deg 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 strength of 3.0 ksi and 4.0 ksi. For Class A concrete (f'_c = 3.5 ksi), use development lengths shown for f'_c = 3.0 ksi unless calculated independently.

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

62-2.07 Splices

Reference: Article 5.11.5

62-2.07(01) General

Lap splices or mechanical splices may be used to splice reinforcing bars: Lap splicing of reinforcing bars is the most common method. The plans should show the locations and lengths of all lap splices. Due to splice lengths required, lap splices are not permitted for #11 bars or larger. However, if #11 bars or larger are necessary, mechanical bar splices should be used. Mechanical bar splices should also be considered in lieu of lap splices in a highly-congested area. Mechanical splices are required for tension tie members.

Lap splices, for either tension or compression bars, should not be less than 12 in. See 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. Tension lap splices are classified as A, B, or C. Bars should be spliced at points of minimum stress.

For a tension splice, the length of a lap splice between bars of different sizes should 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 strength of 3.0 ksi or 4.0 ksi. For class A Concrete (f'_c = 3.5 ksi), use splice lengths shown for f'_c = 3.0 ksi unless calculated independently.

Splice lengths for spacing ≥ 6 in., 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 a compression member are sized for tension lap splices. The design of a compression member, such as a column, pier wall, or abutment wall, 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 mechanical splice is a proprietary splicing mechanism. The requirements for mechanical splices are found in *LRFD* Articles 5.11.5.2.2, 5.11.5.3.2, and 5.11.5.5.2. All mechanical connectors should 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 hook or bend diameters, see Figure 62-2Y. The value of A should be used for a standard 90-deg hook for both longitudinal reinforcement (end hook) and transverse reinforcement (stirrup or tie hook). For transverse reinforcement where the bar size is #3 or #4 and shorter tail lengths are required for better constructability, non-standard hooks may be used. Dimensions and bend diameters of non-standard 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 1 in. 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

References: 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 12 in. 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 footing of each interior substructure unit that is located below a deck expansion joint. For a tall pier or bent, engineering judgment should be exercised;
6. concrete bridge railing;
7. bars extending into the deck from the beams or substructure; or
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; or
2. a reinforced-concrete retaining wall.

62-2.10 Bar Detailing

62-2.10(01) Standard Practice

The following provides the standard practice for detailing reinforcing bars.

1. Reinforcing bars should be called out in the plan, elevation, and sections to clearly indicate the size, location, and spacing of the individual bars. The number of reinforcing bars should 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 should be called out.

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

3. All dimensions on details are measured on centerlines of bars, except where cover, e.g., 2 in. cl., is indicated.

4. Straight bars will be designated by size and length (e.g., #4 x 15'-0").

5. Straight-bar lengths should be in 3-in. multiples, except for short vertical bars in a railing or a parallel wing, which should be in 1-in. 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 may be given a lower-case-letter suffix to indicate the location of the bar in the proper element of the structure (e.g., 801a, 802a). 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.

7. 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. The one letter is to apply to both.

8. Epoxy-coated bars will be suffixed by the letter E (e.g., #6E x 15'-0", 801aE). 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. If rounding off lengths of bars, one length should not encroach upon the minimum clearances.

3. Consideration should 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 provides a section through a hypothetical member showing some of the accepted methods for detailing reinforcing steel. The following list describes some of the concerns and observations that should be considered when detailing reinforcing steel.

A section view should be drawn to a large-enough scale to clearly show reinforcing details.

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

2. Bends of standard hooks and stirrups need not be dimensioned. However, all bends should be drawn to scale.

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

4. An arrowhead pointing to the bar or a circle drawn around the bar are the acceptable methods of detailing for a bar shown end-on. An arrowhead should point directly to the bar.

5. Sections cut at specific locations along a member are preferred to a typical section for a complex reinforcing pattern.

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 is the standard practice for detailing bending diagrams.

1. All dimensions are measured out-to-out of bars.
2. All bent-bar partial dimensions should be shown to the nearer ¼ in.
3. The overall length of a bent bar should be rounded up to the next 1 in.

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 mats. 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 nearer 1 in.
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 inch.
4. The length *L* is the sum of *A* + *B*.

The second method should 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 ,*B*, and shortest, *A*, bars required to the nearer 1 in.
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 *N*. Round the increment to the nearest inch.
4. Determine dimensions *B* and *C* as follows:

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

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. The bars should be listed in descending order of size.

2. For each bar size, bent bars should be listed sequentially by number first followed by straight bars.
3. Straight bars should be listed in descending order of length.
4. Subtotals of the mass should be provided for each bar size.
5. Plain and epoxy-coated bars should be billed separately with totals for each.
6. There should be separate Bills of Materials shown on the appropriate plan sheet 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.

This Section provides information for the design of a reinforced cast-in-place concrete slab superstructure that amplifies or clarifies the requirements of 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 [revised Mar 2009]

Reference: Article 5.12.3

Figure 62-2C provides criteria for minimum concrete cover for all structure elements. All clearances to reinforcing steel should be shown on the 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 proper stress flow.

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. It is preferred for 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 ratio. The ratio between the depths at the centerlines of the interior piers and at the point of maximum positive moment should be between 2.0 and 2.5. Except for aesthetics, 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 or structurally counterproductive.

62-3.01(04) Substructures

The following describes the practice for types of substructures used.

1. <u>End Supports</u>. Where possible, use integral end bents. Their use is not restricted by highway alignment or skew. The maximum bridge length is 200 ft 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 200 ft.

2. <u>Interior Supports</u>. See Section 59-2.03 for 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 in accordance with *LRFD Specifications* Articles

5.7.3.3.2 and 5.10.8. 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 for nearly identical amounts of minimum reinforcement.

According to *LRFD Specifications* Article 5.14.4.1, bottom transverse reinforcement, with the minimum requirements 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 *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 a heavily skewed or curved bridge, the analytical approach is recommended.

Section 62-3.05 provides 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-spans reinforced-concrete slab superstructure, a dead-load deflection diagram showing the quarter-point deflections should be provided on the plans. The contractor uses this information to develop screed elevations that will enable it 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 to be provided at 5-ft 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 deflections values to the nearest 0.10 in.
5. The deflection of the concrete slab caused by the weight of a concrete 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 if concrete placement is interrupted due to rain or other unavoidable event.

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 state, and not the strength or extreme-event limit state. An edge beam can be a thickened section 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.

The required shrinkage and temperature reinforcement, as a function of slab thickness, is provided in Figure 62-3D.

62-3.06 Reinforcing Steel and Constructibility

The following practices for reinforcing-steel placement should be considered to improve the constructability.

1. The maximum reinforcing-bar size should be #11.
2. The minimum spacing of reinforcing bars should preferably be 6 in.
3. Longitudinal steel should be detailed in a 2-bar alternating pattern, with one of the bars continuous through the slab. The maximum size difference should be two standard bar sizes.

Vertical steel, other than that required to keep the longitudinal negative-moment reinforcement floating, may not be required. *LRFD Specifications* Article 5.11.1.2 provides requirements 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 requirement 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 a reinforced-concrete slab bridge only. The drain is a 6-in. PVC pipe set into the deck. The small deck drains have limited hydraulic capacity; therefore, the standard spacing is approximately 6 ft. A 0.5-in. depression, which extends 12 in. transversely from the face of the curb, slightly increases the capacity. The PVC pipe must clear the bent-cap face by 2 ft.

2. Concrete-Railing Scuppers. Scuppers in a concrete railing are permitted only on a local public agency structure.

62-3.08 Distribution of Concrete Railing Dead Load

The dead load of the railing should be assumed to be distributed uniformly over the entire bridge width.

62-3.09 Distribution of Live Loads

Reference: Article 4.6.2.3

Section 60-3.02 discusses the application of vehicular live load. 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*, should 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. *E* is the transverse width of slab over which an axle unit is distributed.

3. Using *LRFD Specifications* Equation 4.6.2.3-3 for the reduction of moments in a skewed structure will not 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 *LRFD Specifications* Article 4.6.2.3 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 *LRFD Specifications* Table 2.5.2.6.3-1. 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 0.5-in. 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

LRFD Specifications Article 5.11.1.2 provides requirements 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 deg, the transverse reinforcement is permitted to be parallel to the skew, providing for equal bar lengths. For a skew angle of 45 deg or greater, the transverse reinforcement should be placed perpendicular to the longitudinal reinforcement. This requirement 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 design or modifications to the integral end supports are not required for a greatly-skewed or -curved structure. The requirements are based upon performance of relatively small span structures constructed to date. Such slab superstructures have included skews in excess of 50 deg and moderate curvatures. A significant deviation from successful past practice should be reviewed. See Figure 62-3C.

62-3.14 Design Requirements for Integral Bent Cap at Slab Superstructure

The following are the requirements for the design of an integral bent cap.

1. The standard pile-cap dimensions are a width of 30 in. and a depth of 18 in. plus the slab thickness.
2. For a skewed structure, the 30-in. width dimension is measured perpendicular to the skew.
3. All transverse reinforcement (stirrups) in the cap is placed perpendicular to the skew.
4. Minimum concrete cover for cap reinforcing steel is 2 in.
5. Standard pile embedment into an end-bent or interior-bent cap is 12 in.
6. Support bars and coping stirrup bars are used to provide support for the top steel in the slab. Stirrup bars should be placed parallel to the skew.
7. A 0.75-in., half-round drip bead should be located under the deck, 6 in. in from the face of coping.

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

1. Either a level or sloped profile can be easily formed.

2. The profile can be made level if the difference in top-of-slab elevations at the left and right copings, along the centerline of the bent cap, is 3 in. or less. For a difference greater than 3 in., slope the bottom of the cap from coping to coping.

Figures 62-3E through 62-3H provide 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 should 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 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, as follows:

$$f_f \le 21 - 0.33 f_{\min} + 8\left(\frac{r}{h}\right)$$

The section properties should be based on a cracked section where the sum of the stresses due to unfactored permanent loads plus 1.5 times the fatigue load is tensile and exceeds $0.95\sqrt{f'_c}$. A section in a stress-reversal area should be analyzed as a doubly-reinforced section.

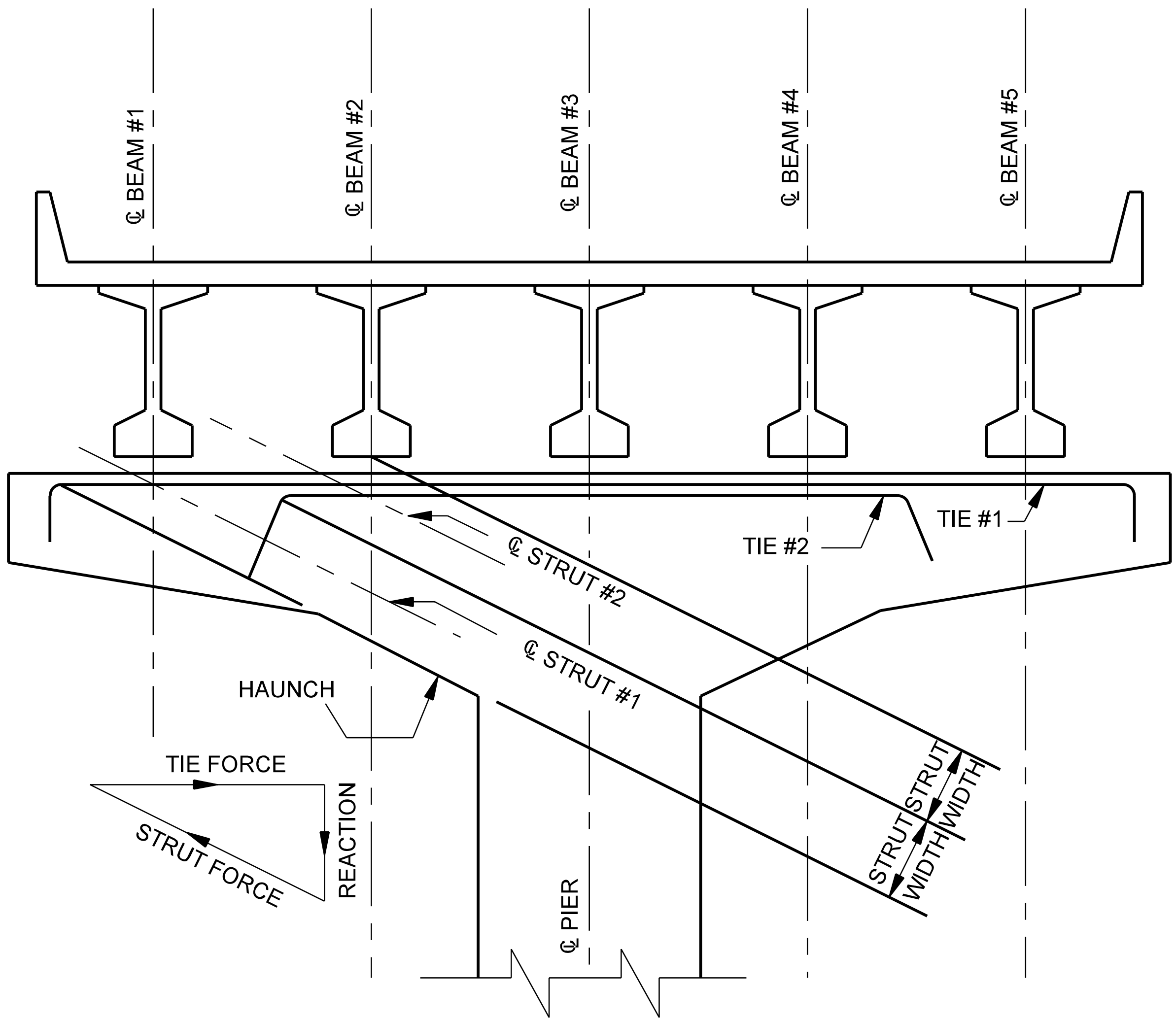
Concrete	Yield Strength, f_c' (psi)	Modulus of Elasticity, E_c (ksi)	Modulus of Rupture, f_r (psi)
Class C	4000	3645	480
Class A	3500	3410	450
Class B	3000	3155	415

Notes:

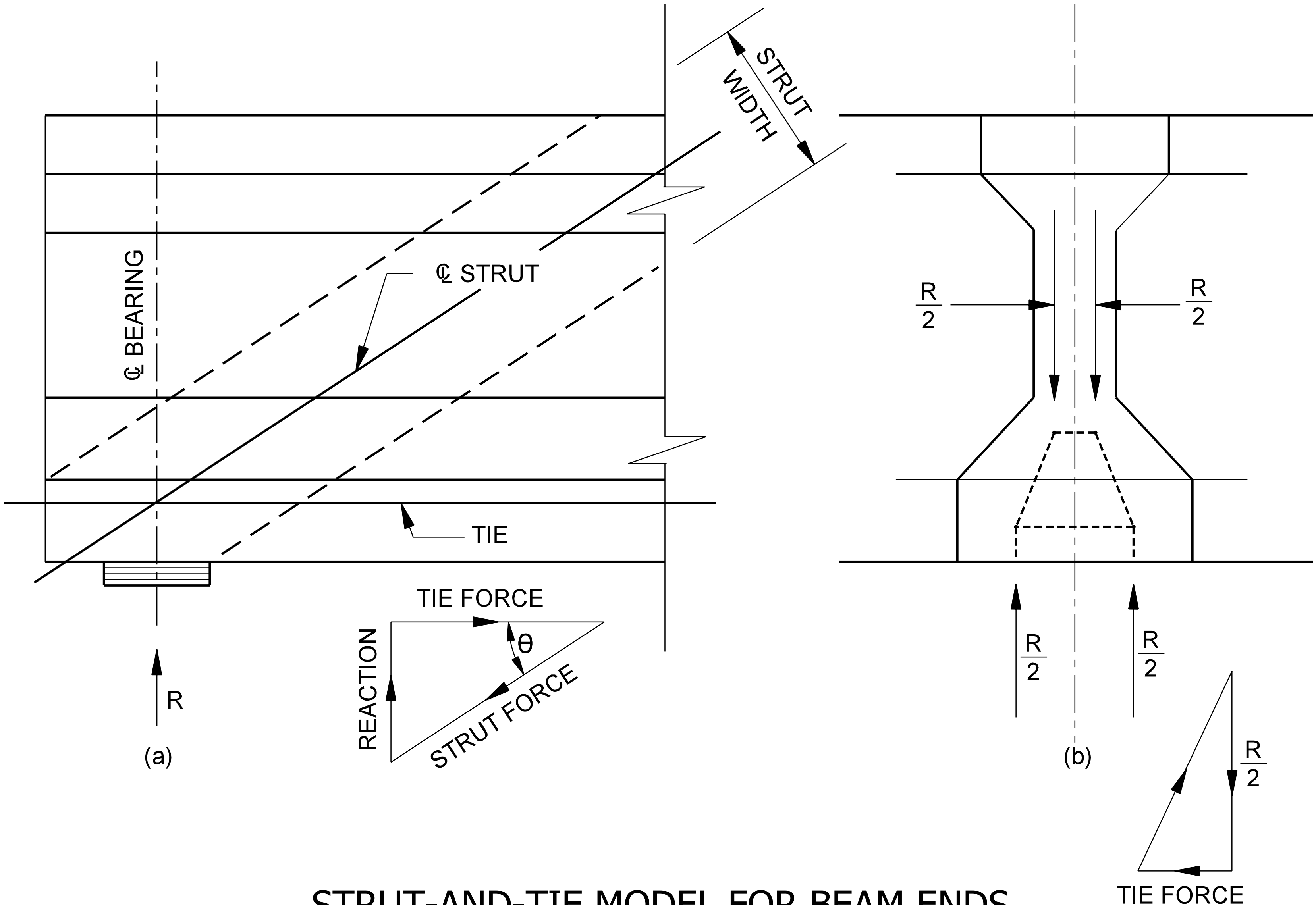
1. *Thermal coefficient of expansion = 6.0 x* $10^{-6}/°F$
2. *Shrinkage coefficient = 0.0002 after 28 days*
   = 0.0005 after 1 year
3. *Normal weight concrete density = 150* lb/ft^3 *for computing loads*
   = 145 lb/ft^3 *for computing properties*

MATERIAL PROPERTIES OF CONCRETE

Figure 62-1A



STRUT-AND-TIE MODEL FOR HAMMERHEAD PIER
Figure 62-1B



STRUT-AND-TIE MODEL FOR BEAM ENDS

Figure 62-1C

Bar Size Designation	Nominal Dimensions			Maximum Bar Length for Fabrication (ft)	Preferred Maximum Bar Length for Detailing (ft)
	Weight (lb/ft)	Diameter (in)	Area (in^2)		
#3*	0.376	0.375	0.11		
#4*	0.668	0.500	0.20	35	30
#5	1.043	0.625	0.31	45	40
#6	1.502	0.750	0.44	45	40
#7	2.044	0.875	0.60	45	40
#8	2.670	1.000	0.70	45	40
#9	3.400	1.128	1.00	45	40
#10	4.303	1.270	1.27	45	40
#11	5.313	1.410	1.56	45	40
#14	7.650	1.693	2.25	45	40
#18	13.600	2.257	4.00	45	40

Note: *Maximum bar length does not apply to spiral bars.*

REINFORCING-BAR SIZES

Figure 62-2A

Bar Size	Area (in.^{2})	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
#3	0.11	0.33	0.26	0.22	0.19	0.17	0.15	0.13	0.12	0.11	0.10	0.09	0.09	0.08	0.08	0.07
#4	0.20	0.60	0.48	0.40	0.34	0.30	0.27	0.24	0.22	0.20	0.18	0.17	0.16	0.15	0.14	0.13
#5	0.31	0.93	0.74	0.62	0.53	0.47	0.41	0.37	0.34	0.31	0.29	0.27	0.25	0.23	0.22	0.21
#6	0.44	1.32	1.06	0.88	0.75	0.66	0.59	0.53	0.48	0.44	0.41	0.38	0.35	0.33	0.31	0.29
#7	0.60	1.80	1.44	1.20	1.03	0.90	0.80	0.72	0.65	0.60	0.55	0.51	0.48	0.45	0.42	0.40
#8	0.70	2.10	1.68	1.40	1.20	1.05	0.93	0.84	0.76	0.70	0.65	0.60	0.56	0.53	0.49	0.47
#9	1.00	3.00	2.40	2.00	1.71	1.50	1.33	1.20	1.09	1.00	0.92	0.86	0.80	0.75	0.71	0.67
#10	1.27	3.81	3.05	2.54	2.18	1.91	1.69	1.52	1.39	1.27	1.17	1.09	1.02	0.95	0.90	0.85
#11	1.56	4.68	3.74	3.12	2.67	2.34	2.08	1.87	1.70	1.56	1.44	1.34	1.25	1.17	1.10	1.04
#14	2.25	6.75	5.40	4.50	3.86	3.38	3.00	2.70	2.45	2.25	2.08	1.93	1.80	1.39	1.59	1.50
#18	4.00	12.0	9.60	8.00	6.86	6.00	5.33	4.80	4.36	4.00	3.69	3.43	3.20	3.00	2.82	2.67

REINFORCING BARS
Area (in.^{2}) Per One-Foot Section

Figure 62-2B

Item	Cover
Deck or Reinforced-Concrete Slab:	
Top Bars	2½ *
Bottom Bars	1
Ends of Slab	2
Faces of Copings	2
Footing:	
General	3
Bottom Bars	4
Columns, Ties, and Stirrups	1½
All Other Structural Elements	2

** Includes a ½-in. sacrificial wearing surface.*

MINIMUM CONCRETE COVER (in.)
FOR DESIGN AND DETAILING

Figure 62-2C

Bar Size	Minimum Center-to-Center Spacing	
	Unspliced Bars	Spliced Bars
#3	2	2¼
#4	2	2½
#5	2¼	2¾
#6	2¼	3
#7	2½	3¼
#8	2½	3½
#9	3¼	4
#10	3½	4½
#11	3¾	5
#14	4¼	n/a
#18	5¾	n/a

Note: *Minimum spacing value, rounded up to the nearest 1/4 in., should be based on LRFD Specifications Articles 5.10.3.1.1 and 5.10.3.1.4. The maximum size of coarse aggregate used in cast-in-place or precast concrete is 1 in.*

MINIMUM CENTER-TO-CENTER SPACING OF BARS (in.)

Figure 62-2D

Bar Size	Area (in.^{2})	Top Bars (1)	Others (2)	Top Bars (3)	Others (4)
#3	0.11	1’-1”	1’-0”	1’-0”	1’-0”
#4	0.2	1’-5”	1’-0”	1’-2”	1’-0”
#5	0.31	1’-9”	1’-3”	1’-5”	1’-0”
#6	0.44	2’-3”	1’-8”	1’-10”	1’-4”
#7	0.6	3’-1”	2’-2”	2’-6”	1’-9”
#8	0.79	4’-0”	2’-11”	3’-3”	2’-4”
#9	1	5’-1”	3’-8”	4’-1”	2’-11”
#10	1.27	6’-5”	4’-7”	5’-2”	3’-8”
#11	1.56	7’-11”	5’-8”	6’-4”	4’-7”
#14	2.25	10’-11”	7’-10”	8’-9”	6’-3”
#18	4	14’-2”	10’-2”	11’-4”	8’-1”

Notes:

(1) 1.4 x l_d

(2) l_d

(3) 1.4 x 0.8 x l_d

(4) 0.8 x l_d

5. Development length is modified for ≥ 6 in. spacing and minimum 3 in. clear spacing between bars.
6. d_b < Cover; $2d_b$ < Clear Spacing
7. Value is for normal-weight concrete.
8. Top bars are horizontal bars with more than 12 in. of fresh concrete below the reinforcement.

DEVELOPMENT LENGTH FOR UNCOATED BARS IN TENSION

f_c’ = 3 ksi

Figure 62-2E

Bar Size	Area (in.^{2})	Top Bars (1)	Others (2)	Top Bars (3)	Others (4)
#3	0.11	1’-1”	1’-0”	1’-0”	1’-0”
#4	0.2	1’-5”	1’-0”	1’-2”	1’-0”
#5	0.31	1’-9”	1’-3”	1’-5”	1’-0”
#6	0.44	2’-2”	1’-6”	1’-9”	1’-3”
#7	0.6	2’-8”	1’-11”	2’-2”	1’-6”
#8	0.79	3’-6”	2’-6”	2’-10”	2’-0”
#9	1	4’-5”	3’-2”	3’-6”	2’-6”
#10	1.27	5’-7”	4’-0”	4’-6”	3’-3”
#11	1.56	6’-10”	4’-11”	5’-6”	3’-11”
#14	2.25	9’-6”	6’-9”	7’-7”	5’-5”
#18	4	12’-3”	8’-9”	9’-10”	7’-0”

Notes:

(1) 1.4 x l_d

(2) l_d

(3) 1.4 x 0.8 x l_d

(4) 0.8 x l_d

5. Development length is modified for ≥ 6-in. spacing and minimum 3-in. clear spacing between bars.
6. d_b < Cover; 2d_b < Clear Spacing
7. Value is for normal-weight concrete.
8. Top bars are horizontal bars with more than 12 in. of fresh concrete below the reinforcement.

DEVELOPMENT LENGTH FOR UNCOATED BARS IN TENSION

f_c’ = 4 ksi

Figure 62-2F

Bar Size	Area (in.^{2})	Top Bars (1)	Others (2)	Top Bars (3)	Others (4)
#3	0.11	1'-4"	1'-2"	1'-1"	1'-0"
#4	0.2	1'-9"	1'-6"	1'-5"	1'-3"
#5	0.31	2'-2"	1'-11"	1'-9"	1'-6"
#6	0.44	2'-9"	2'-5"	2'-2"	1'-11"
#7	0.6	3'-9"	3'-3"	3'-0"	2'-8"
#8	0.79	4'-11"	4'-4"	3'-11"	3'-6"
#9	1	6'-2"	5'-5"	4'-11"	4'-4"
#10	1.27	7'-10"	6'-11"	6'-3"	5'-6"
#11	1.56	9'-7"	8'-6"	7'-8"	6'-10"
#14	2.25	13'-4"	11'-9"	10'-8"	9'-5"
#18	4	17'-3"	15'-2"	13'-9"	12'-2"

Notes:

(1) 1.7 x l_d

(2) 1.5 x l_d

(3) 1.7 x 0.8 x l_d

(4) 1.5 x 0.8 x l_d

5. Development length is modified for ≥ 6-in. spacing and minimum 3-in. clear spacing between bars.
6. d_b < Cover; 2d_b < Clear Spacing
7. Value is for normal-weight concrete.
8. Top bars are horizontal bars with more than 12 in. of fresh concrete below the reinforcement.

DEVELOPMENT LENGTH FOR EPOXY-COATED BARS IN TENSION

f_c' = 3 ksi

Figure 62-2G

Bar Size	Area (in.^{2})	Top Bars (1)	Others (2)	Top Bars (3)	Others (4)
#3	0.11	1’-4”	1’-2”	1’-1”	1’-0”
#4	0.2	1’-9”	1’-6”	1’-5”	1’-3”
#5	0.31	2’-2”	1’-11”	1’-9”	1’-6”
#6	0.44	2’-7”	2’-3”	2’-1”	1’-10”
#7	0.6	3’-3’	2’-10”	2’-7”	2’-3”
#8	0.79	4’-3”	3’-9”	3’-5”	3’-0”
#9	1	5’-4”	4’-9”	4’-3”	3’-9”
#10	1.27	6’-9”	6’-0”	5’-5”	4’-10”
#11	1.56	8’-4”	7’-4”	6’-8”	5’-11”
#14	2.25	11’-6”	10’-2”	9’-3”	8’-2”
#18	4	14’-11”	13’-2”	11’-11”	10’-6”

Notes:

(1) 1.7 x l_d

(2) 1.5 x l_d

(3) 1.7 x 0.8 x l_d

(4) 1.5 x 0.8 x l_d

5. Development length is modified for ≥ 6-in. spacing and minimum 3-in. clear spacing between bars.
6. d_b < Cover; $2d_b$ < Clear Spacing
7. Value is for normal-weight concrete.
8. Top bars are horizontal bars with more than 12 in. of fresh concrete below the reinforcement.

DEVELOPMENT LENGTH FOR EPOXY-COATED BARS IN TENSION

f_c’ = 4 ksi

Figure 62-2H

Bar Size	l_{dh} Side Cover < 2.5 in., or Cover on Tail < 2 in. $l_{dh} = l_{hb}$	l_{dh} Side Cover ≥ 2.5 in., or Cover on Tail ≥ 2 in. $l_{dh} = 0.7\ l_{hb}$
#3	9”	6”
#4	11”	8”
#5	1’-2”	10”
#6	1’-5”	1’-0”
#7	1’-8”	1’-2”
#8	1’-10”	1’-4”
#9	2’-1”	1’-6”
#10	2’-4”	1’-8”
#11	2’-7”	1’-10”
#14	3’-2”	3’-2”
#18	4’-2”	4’-2”

* $l_{dh} = l_{hb}$

HOOKED UNCOATED-BAR DEVELOPMENT LENGTH
f_c’ = 3 ksi

Figure 62-2 I

Bar Size	l_{dh} Side Cover < 2.5 in., or Cover on Tail < 2 in. $l_{dh} = l_{hb}$	l_{dh} Side Cover ≥ 2.5 in., or Cover on Tail ≥ 2 in.. $l_{dh} = 0.7\ l_{hb}$
#3	8”	6”
#4	10”	7”
#5	1’-0”	9”
#6	1’-3”	10”
#7	1’-5”	1’-0”
#8	1’-7”	1’-2”
#9	1’-10”	1’-4”
#10	2’-1”	1’-5”
#11	2’-3”	1’-7”
#14	2’-9”	2’-9”
#18	3’-7”	3’-7”

* $l_{dh} = l_{hb}$

HOOKED UNCOATED-BAR DEVELOPMENT LENGTH

f_c’ = 4 ksi

Figure 62-2J

Bar Size	l_{dh} Side Cover < 2.5 in., or Cover on Tail < 2 in. $l_{dh} = l_{hb}$	l_{dh} Side Cover ≥ 2.5 in., or Cover on Tail ≥ 2 in. $l_{dh} = 0.7\ l_{hb}$
#3	10”	7”
#4	1’-2”	10”
#5	1’-5”	1’-0”
#6	1’-8”	1’-2”
#7	2’-0”	1’-5”
#8	2’-3”	1’-7”
#9	2’-6”	1’-9”
#10	2’-10”	2’-0”
#11	3’-2”	2’-2”
#14	3’-9”	3’-9”
#18	5’-0”	5’-0”

* $l_{dh} = l_{hb}$

HOOKED EPOXY-COATED-BAR DEVELOPMENT LENGTH
f_c’ = 3 ksi

Figure 62-2K

Bar Size	l_{dh} Side Cover < 2.5 in., or Cover on Tail < 2 in. $l_{dh} = l_{hb}$	l_{dh} Side Cover ≥ 2.5 in., or Cover on Tail ≥ 2 in. $l_{dh} = 0.7\ l_{hb}$
#3	9”	6”
#4	1’-0”	8”
#5	1’-3”	10”
#6	1’-6”	1’-0”
#7	1’-8”	1’-2”
#8	1’-11”	1’-4”
#9	2’-2”	1’-7”
#10	2’-5”	1’-9”
#11	2’-9”	1’-11”
#14	3’-3”	3’-3’
#18	4’-4”	4’-4”

* $l_{dh} = l_{hb}$

HOOKED EPOXY-COATED-BAR DEVELOPMENT LENGTH
f_c’ = 4 ksi

Figure 62-2L

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1'-1"	1'-0"	1'-0"	1'-0"
#4	1'-5"	1'-0"	1'-2"	1'-0"
#5	1'-9"	1'-3"	1'-5"	1'-0"
#6	2'-3"	1'-8"	1'-10"	1'-4"
#7	3'-1"	2'-2"	2'-6"	1'-9"
#8	4'-0"	2'-11"	3'-3"	2'-4"
#9	5'-1"	3'-8"	4'-1"	2'-11"
#10	6'-5"	4'-7"	5'-2"	3'-8"
#11	7'-11"	5'-8"	6'-4"	4'-7"

Notes:

1. All splice lengths in feet and inches
2. d_b < Cover
3. $2d_b$ < Clear Spacing
4. Values are for normal weight concrete.

CLASS A SPLICE LENGTH FOR UNCOATED BARS IN TENSION
f_c' = 3 ksi

Figure 62-2M

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-1”	1’-0”	1’-0”	1’-0”
#4	1’-5”	1’-0”	1’-2”	1’-0”
#5	1’-9”	1’-3”	1’-5”	1’-0”
#6	2’-2”	1’-6”	1’-9”	1’-3”
#7	2’-8”	1’-11”	2’-2”	1’-6”
#8	3’-6”	2’-6”	2’-10”	2’-0”
#9	4’-5”	3’-2”	3’-6”	2’-6”
#10	5’-7”	4’-0”	4’-6”	3’-3”
#11	6’-10”	4’-11”	5’-6”	3’-11”

Notes:

1. d_b < Cover
2. $2d_b$ < Clear Spacing
3. Value is for normal-weight concrete.

CLASS A SPLICE LENGTH FOR UNCOATED BARS IN TENSION

f_c’ = 4 ksi

Figure 62-2N

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-4”	1’-2”	1’-1”	1’-0”
#4	1’-9”	1’-6”	1’-5”	1’-3”
#5	2’-2”	1’-11”	1’-9”	1’-6”
#6	2’-9”	2’-5”	2’-2”	1’-11’
#7	3’-9”	3’-3”	3’-0”	2’-8”
#8	4’-11”	4’-4”	3’-11”	3’-6”
#9	6’-2”	5’-5”	4’-11”	4’-4”
#10	7’-10”	6’-11”	6’-3”	5’-6”
#11	9’-7”	8’-6”	7’-8”	6’-10”

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS A SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c’ = 3 ksi

Figure 62-2 O

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-4”	1’-2”	1’-1”	1’-0”
#4	1’-9”	1’-6”	1’-5”	1’-3”
#5	2’-2”	1’-11”	1’-9”	1’-6”
#6	2’-7”	2’-3”	2’-1”	1’-10”
#7	3’-3”	2’-10”	2’-7”	2’-3”
#8	4’-3”	3’-9”	3’-5”	3’-0”
#9	5’-4”	4’-9”	4’-3”	3’-9”
#10	6’-9”	6’-0”	5’-5”	4’-10”
#11	8’-4”	7’-4”	6’-8”	5’-11”

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS A SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c’ = 4 ksi

Figure 62-2P

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-5”	1’-0”	1’-2”	1’-0”
#4	1’-10”	1’-4”	1’-6”	1’-1”
#5	2’-4”	1’-8”	1’-10”	1’-4”
#6	2’-11”	2’-1”	2’-4”	1’-8”
#7	4’-0”	2’-10”	3’-2”	2’-4”
#8	5’-3”	3’-9”	4’-2”	3’-0”
#9	6’-7”	4’-9”	5’-4”	3’-10”
#10	8’-5”	6’-0”	6’-9”	4’-10”
#11	10’-3”	7’-4”	8’-3”	5’-11”

Notes:

1. d_b < Cover
2. $2d_b$ < Clear Spacing
3. Value is for normal-weight concrete.

CLASS B SPLICE LENGTH FOR UNCOATED BARS IN TENSION

f_c’ = 3 ksi

Figure 62-2Q

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1'-5"	1'-0"	1'-2"	1'-0"
#4	1'-10"	1'-4"	1'-6"	1'-1"
#5	2'-4"	1'-8"	1'-10"	1'-4"
#6	2'-9"	2'-0"	2'-3"	1'-7"
#7	3'-5"	2'-6"	2'-9"	2'-0"
#8	4'-6"	3'-3"	3'-8"	2'-7"
#9	5'-9"	4'-1"	4'-7"	3'-3"
#10	7'-3"	5'-2"	5'-10"	4'-2"
#11	8'-11"	6'-5"	7'-2"	5'-1"

Notes:

1. d_b < Cover
2. $2d_b$ < Clear Spacing
3. Value is for normal-weight concrete.

CLASS B SPLICE LENGTH FOR UNCOATED BARS IN TENSION

f_c' = 4 ksi

Figure 62-2R

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-8”	1’-6”	1’-4”	1’-3”
#4	2’-3”	2’-0”	1’-10”	1’-7”
#5	2’-10”	2’-6”	2’-3”	2’-0”
#6	3’-7”	3’-2”	2’-10”	2’-6”
#7	4’-10”	4’-3”	3’-10”	3’-5”
#8	6’-4”	5’-7”	5’-1”	4’-6”
#9	8’-0”	7’-1”	6’-5”	5’-8”
#10	10’-2”	9’-0”	8’-2”	7’-2”
#11	12’-6”	11’-0”	10’-0”	8’-10”

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS B SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c’ = 3 ksi

Figure 62-2S

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-8”	1’-6”	1’-4”	1’-3”
#4	2’-3”	2’-0”	1’-10”	1’-7”
#5	2’-10”	2’-6”	2’-3”	2’-0”
#6	3’-4”	3’-0”	2’-8”	2’-5”
#7	4’-2”	3’-8”	3’-4”	3’-0”
#8	5’-6”	4’-10”	4’-5”	3’-11”
#9	6’-11”	6’-2”	5’-7”	4’-11”
#10	8’-10”	7’-9”	7’-1”	6’-3”
#11	10’-10”	9’-7”	8’-8”	7’-8”

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS B SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c’ = 4 ksi

Figure 62-2T

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-10”	1’-4”	1’-6”	1’-1”
#4	2’-5”	1’-9”	1’-11”	1’-5”
#5	3’-0”	2’-2”	2’-5”	1’-9”
#6	3’-10”	2’-9”	3’-1”	2’-2”
#7	5’-2”	3’-9”	4’-2”	3’-0”
#8	6’-10”	4’-11”	5’-6”	3’-11”
#9	8’-8”	6’-2”	6’-11”	4’-11”
#10	10’-11”	7’-10”	8’-9”	6’-3”
#11	13’-5”	9’-7”	10’-9”	7’-8”

Notes:

1. d_b < Cover
2. $2d_b$ < Clear Spacing
3. Value is for normal-weight concrete.

CLASS C SPLICE LENGTH FOR UNCOATED BARS IN TENSION
f_c’ = 3 ksi

Figure 62-2U

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	1’-10”	1’-4”	1’-6”	1’-1”
#4	2’-5”	1’-9”	1’-11”	1’-5”
#5	3’-0”	2’-2”	2’-5”	1’-9”
#6	3’-7”	2’-7”	2’-11”	2’-1”
#7	4’-6”	3’-3”	3’-7”	2’-7”
#8	5’-11”	4’-3”	4’-9”	3’-5”
#9	7’-6”	5’-4”	6’-0”	4’-3”
#10	9’-6”	6’-9”	7’-7”	5’-5”
#11	11’-8”	8’-4”	9’-4”	6’-8”

Notes:

1. d_b < Cover
2. $2d_b$ < Clear Spacing
3. Value is for normal-weight concrete.

CLASS C SPLICE LENGTH FOR UNCOATED BARS IN TENSION
f_c’ = 4 ksi

Figure 62-2V

Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	2'-3"	1'-11"	1'-9"	1'-7"
#4	2'-11"	2'-7"	2'-4"	2'-1"
#5	3'-8"	3'-3"	2'-11"	2'-7"
#6	4'-8"	4'-1"	3'-9"	3'-3"
#7	6'-4"	5'-7"	5'-1"	4'-6"
#8	8'-3"	7'-4"	6'-8"	5'-10"
#9	10'-6"	9'-3"	8'-5"	7'-5"
#10	13'-3"	11'-9"	10'-8"	9'-5"
#11	16'-4"	14'-5"	13'-1"	11'-6"

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS C SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c' = 3 ksi

Figure 62-2W

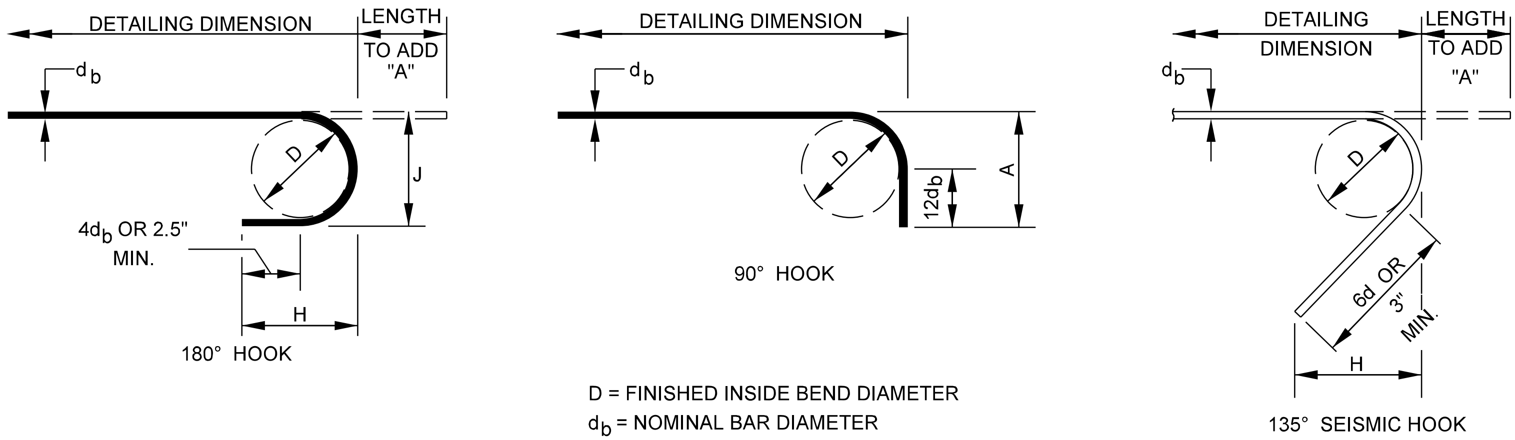
Bar Size	Center to Center Spacing < 6 in., or Cover < 3 in.		Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.	
	Top Bars	Others	Top Bars	Others
#3	2’-3”	1’-11”	1’-9”	1’-7”
#4	2’-11”	2’-7”	2’-4”	2’-1”
#5	3’-8”	3’-3”	2’-11”	2’-7”
#6	4’-5”	3’-10”	3’-6”	3’-1”
#7	5’-6”	4’-10”	4’-5”	3’-10”
#8	7’-2”	6’-4”	5’-9”	5’-1”
#9	9’-1”	8’-0”	7’-3”	6’-5”
#10	11’-6”	10’-2”	9’-3”	8’-2”
#11	14’-2”	12’-6”	11’-4”	10’-0”

Notes:

1. $d_b \leq$ Cover $< 3d_b$
2. $2d_b \leq$ Clear Spacing $< 6d_b$
3. Value is for normal-weight concrete.

CLASS C SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION
f_c’ = 4 ksi

Figure 62-2X



RECOMMENDED END HOOKS, ALL GRADES					
BAR SIZE	D	180° HOOKS			90° HOOKS
		A	J	H	A
#3	2"	5"	2.75"	4"	6"
#4	3"	6"	4"	4.5"	8"
#5	4"	7"	5.25"	5"	10"
#6	4.5"	8"	6"	6"	1'-0"
#7	5.5"	10"	7.25"	7"	1'-3"
#8	6"	11"	8"	8"	1'-5"
#9	10"	1'-3"	1'-0.25"	10"	1'-7"
#10	11"	1'-5"	1'-1.5"	11.5"	1'-10"
#11	1'-0"	1'-7"	1'-3"	1'-0.5"	2'-0"
#14	1'-6.3"	2'-3"	1'-9.5"	1'-5"	2'-7"
#18	2'-0"	3'-1"	2'-4.5"	1'-10.5"	3'-6"

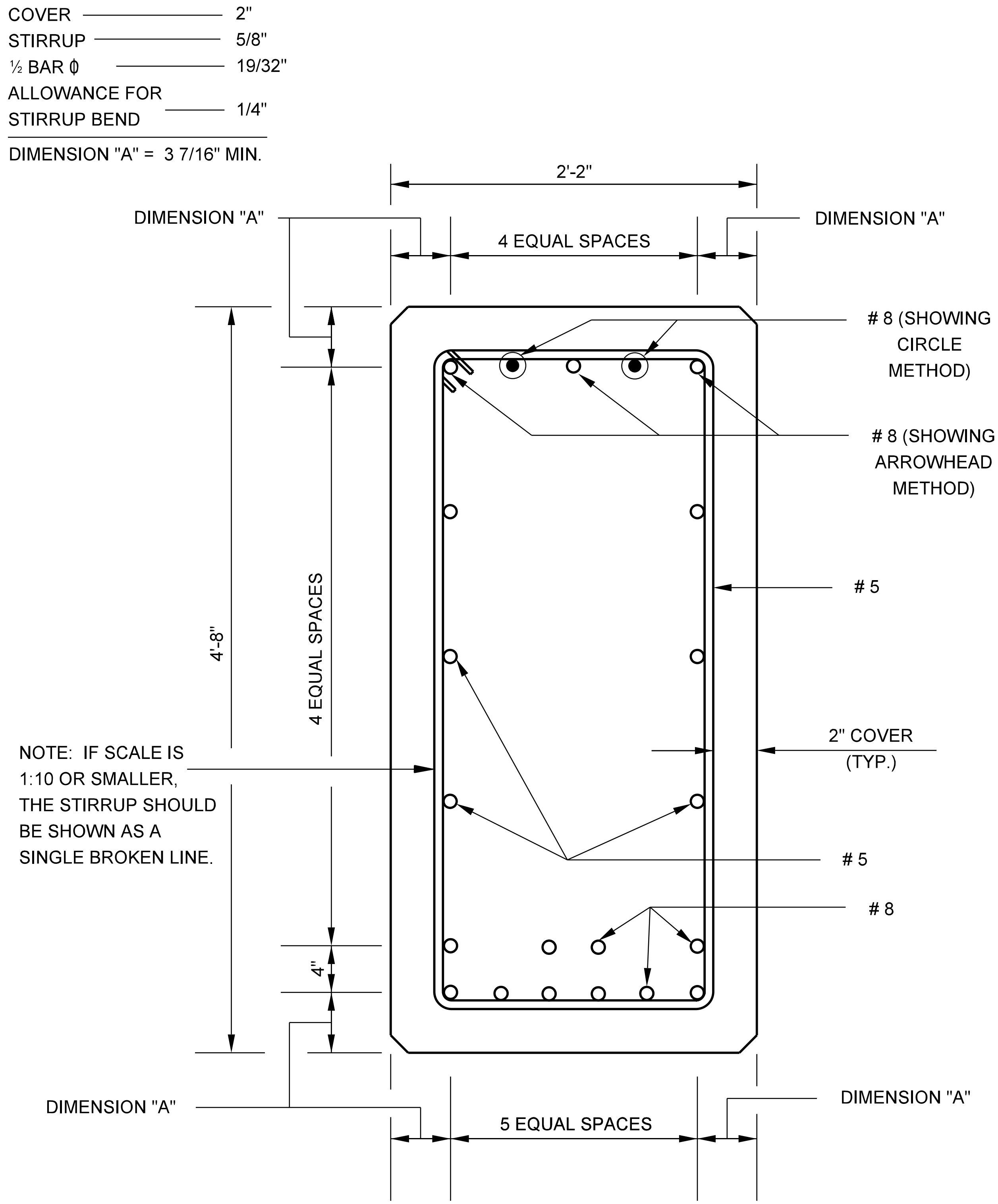
SEISMIC TIE HOOKS			
BAR SIZE	135° SEISMIC HOOKS		
	A	J	H
#3	1.5"	4.25"	3"
#4	2"	4.5"	3.5"
#5	2.5"	5.5"	3.75"
#6	4.5"	8"	4.5"
#7	5.5"	9"	6"
#8	6"	10.5"	6"

Notes:

1. Show detailing dimension and total length of bent bar on the bending diagram in the plans.
   Do not show length to add (dimension "A") for 180 hooks or 135 seismic hooks. Do not show bend diameter unless it is not standard.
2. In computing total length of a bent bar with 90 hooks, do not deduct for bends.

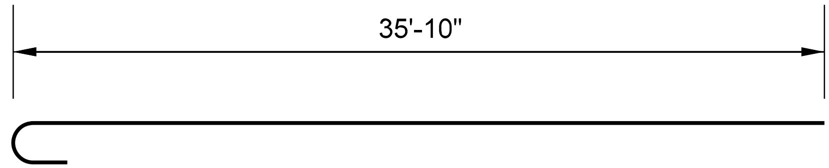
HOOKS AND BENDS

Figure 62-2Y



BARS IN SECTION
Figure 62-2Z

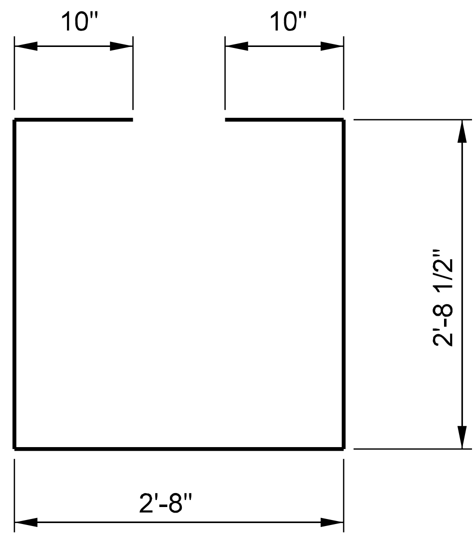
EXAMPLE NO. 1



802c X 36'-9"

35'-10"
11" (180° HOOK)
36'-9"

EXAMPLE NO. 2

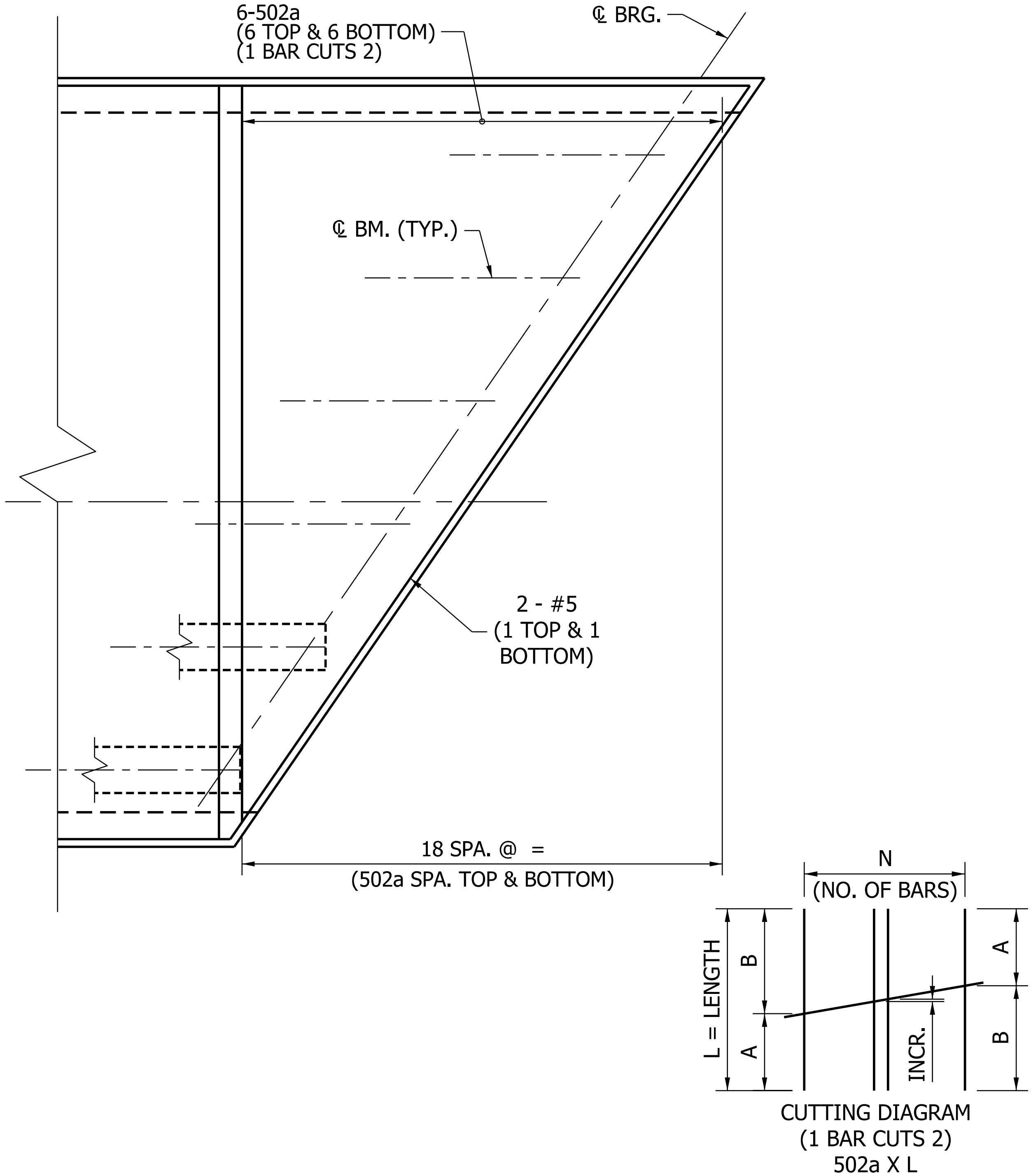


502c X 9'-9"

2'-8 1/2"
2'-8 1/2"
2'-8"
10" (90° HOOK)
10" (90° HOOK)
9'-9"

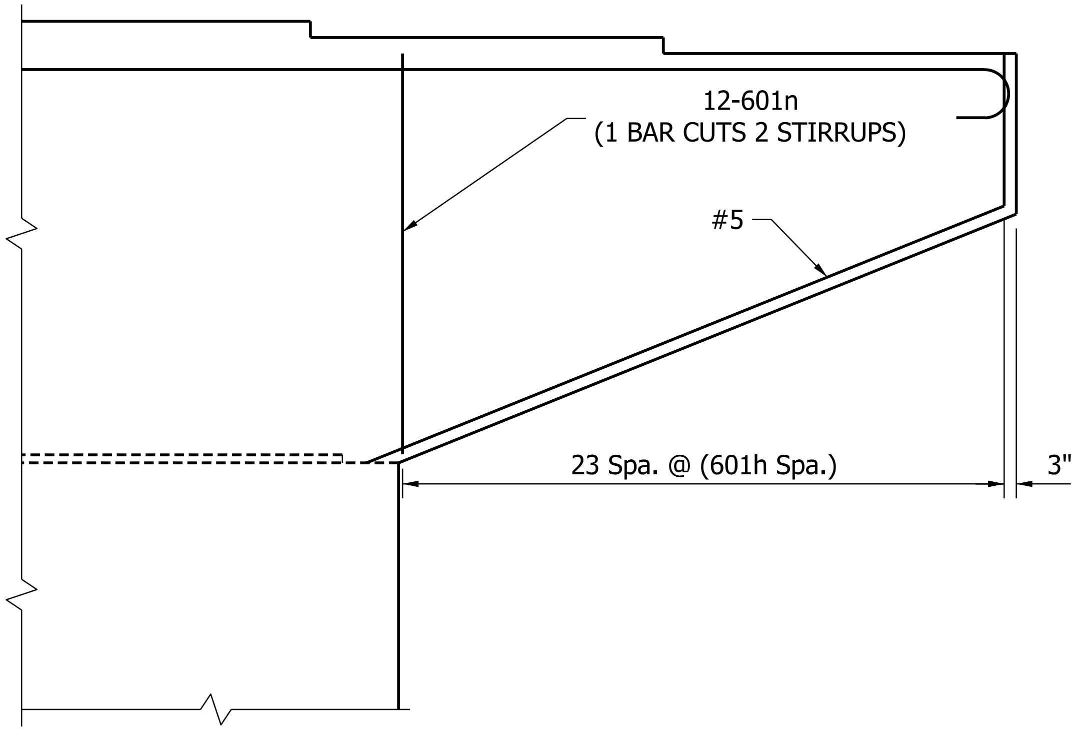
BENDING DIAGRAM EXAMPLES

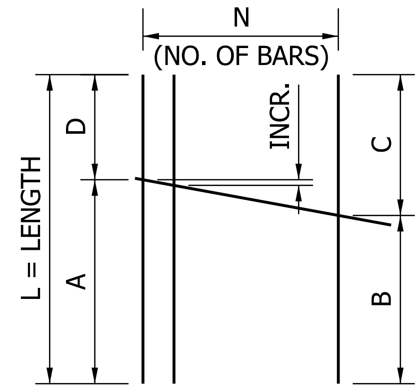
Figure 62-2AA



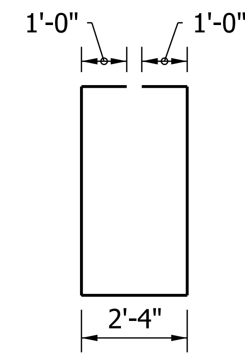
CUTTING DIAGRAM
(Transverse Steel in Bridge Deck)

Figure 62-2BB





CUTTING DIAGRAM



BENDING DIAGRAM

601n X L
(1 BAR CUTS 2 STIRRUPS)

NOTE:
A CUTTING DIAGRAM CAN ALSO BE USED WHEN STIRRUPS ARE PLACED AT TWO DIFFERENT SPACINGS WITH TWO SEPARATE BAR MARKS. "NO. OF BARS" AND CUTTING DIAGRAM DIMENSIONS FOR EACH BAR MARK CAN BE SHOWN IN A TABLE.

CUTTING DIAGRAM
(Hammerhead Stem Pier)

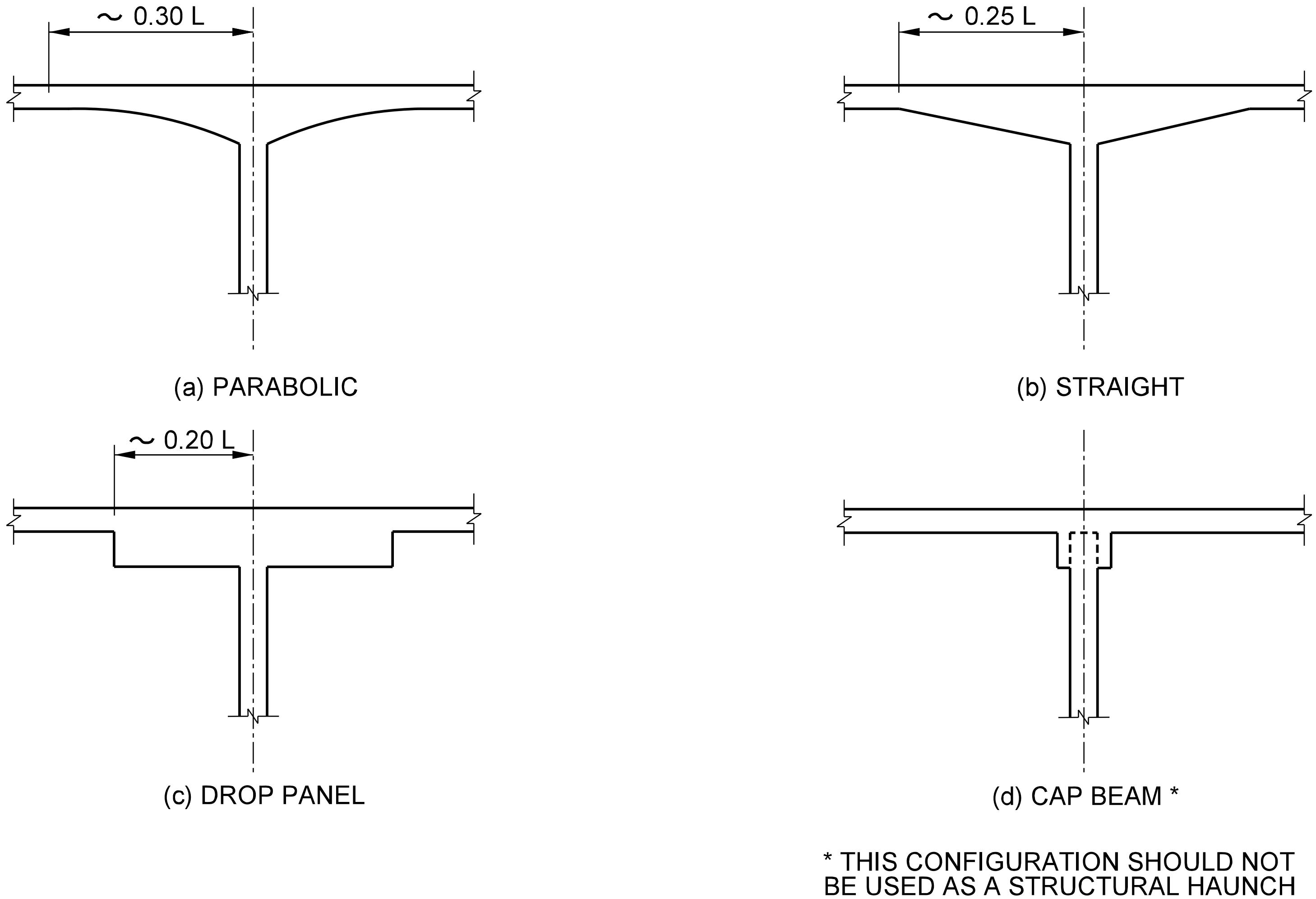
Figure 62-2CC

R.C. BRIDGE APPROACH
BILL OF MATERIALS

Plain Reinforcing Steel			
Size and Mark	No. of Bars	Length	Weight (lb)
503	69	19'-10"	
591	144	20'-7"	
#5	48	31'-0"	
#5	54	29'-4"	
#5	2	26'-0"	
#5	1	24'-8"	
#5	2	22'-0"	
#5	1	20'-8"	
#5	2	18'-4"	
#5	49	16'-8"	
#5	2	14'-4"	
#5	1	12'-8"	
#5	2	10'-4"	
#5	1	9'-0"	
#5	2	6'-4"	
#5	1	5'-0"	
Total No. 5			8853
401	14	3'-7"	
#4	2	19'-8"	
Total No. 4			63
Total Plain Reinforcing Steel			8916
Epoxy-Coated Reinforcing Steel			
#8	4	19'-8"	210
#7	4	19'-8"	161
502	5	20'-2"	
581	51	6'-9"	
591a	31	5'-7"	
593	62	4'-2"	
Total No. 5			914
Total Epoxy-Coated Reinforcing Steel			1285
Concrete			
Reinf.-Conc. Bridge Appr., 15			2950 ft^2
Concrete Railing Class C			2.5 yd^2

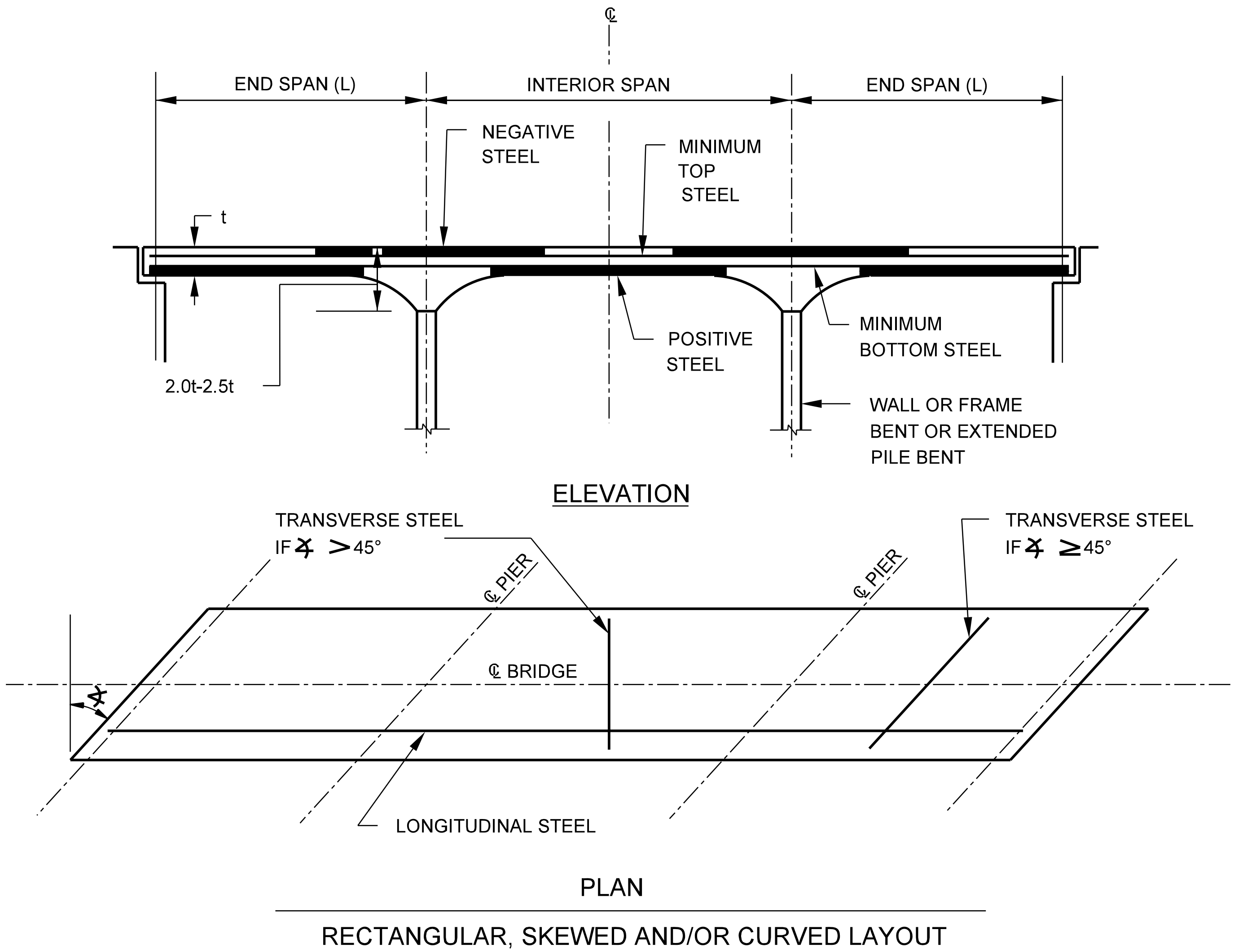
REINFORCED-CONCRETE BRIDGE APPROACH BILL OF MATERIALS

Figure 62-2DD



HAUNCH CONFIGURATIONS FOR REINFORCED CONCRETE SLAB SUPERTRUCTURES

Figure 62-3B



TYPICAL REINFORCED CONCRETE SLAB SUPERSTRUCTURE

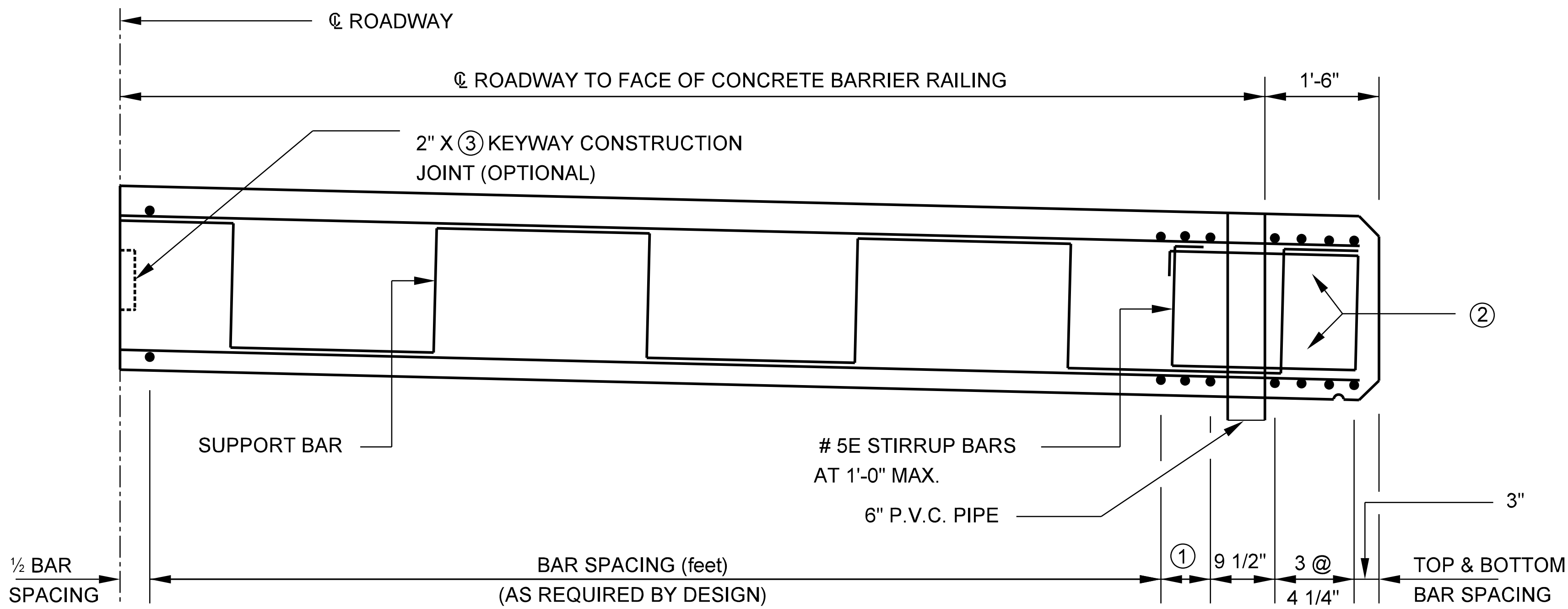
Figure 62-3C

Slab Thickness (in.)	Reinforcement (Top and Bottom)
< 18 18 ≤ Thick. ≤ 28 > 28	#5 @ 1'-6" or #4 @ 1'-0" #5 @ 1'-0" or #4 @ 8" Design per *LRFD* Article 5.10.8.2

SHRINKAGE AND TEMPERATURE REINFORCEMENT FOR SLAB SUPERSTRUCTURE

Figure 62-3D

NOTE: ENTIRE TYPICAL SECTION TO BE DETAILED ON PLANS.



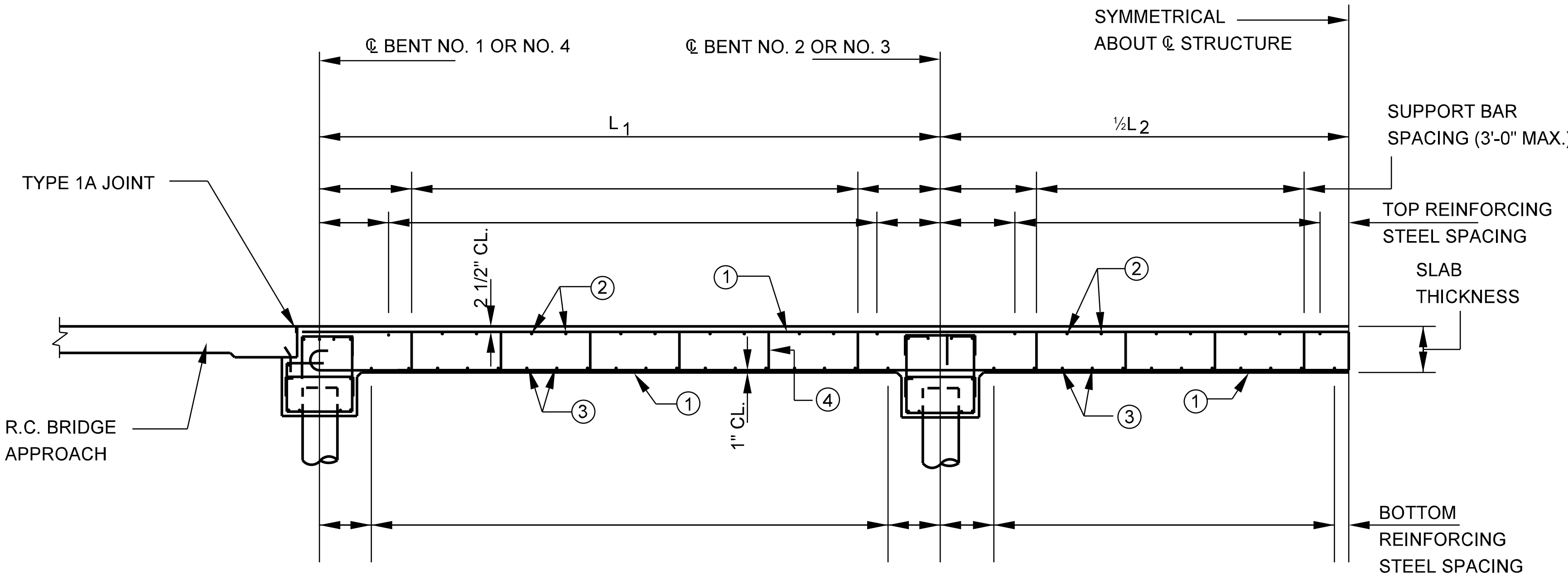
① BAR SPACING AND NUMBER OF SPACES TO BE DETERMINED TO FACILITATE A CONSTANT BAR SPACING IN REMAINDER OF SLAB.

② DESIGN EDGE BEAM IN ACCORDANCE WITH ARTICLES IN THE LRFD SPECIFICATIONS, BUT USE AS A MINIMUM THE SAME AREA OF STEEL PER METER AS IN SLAB.

③ FOR DEPTH OF KEYWAY, USE ONE-THIRD THE SLAB THICKNESS.

INTEGRAL CAPS AT SLAB SUPERSTRUCTURE

Figure 62-3E

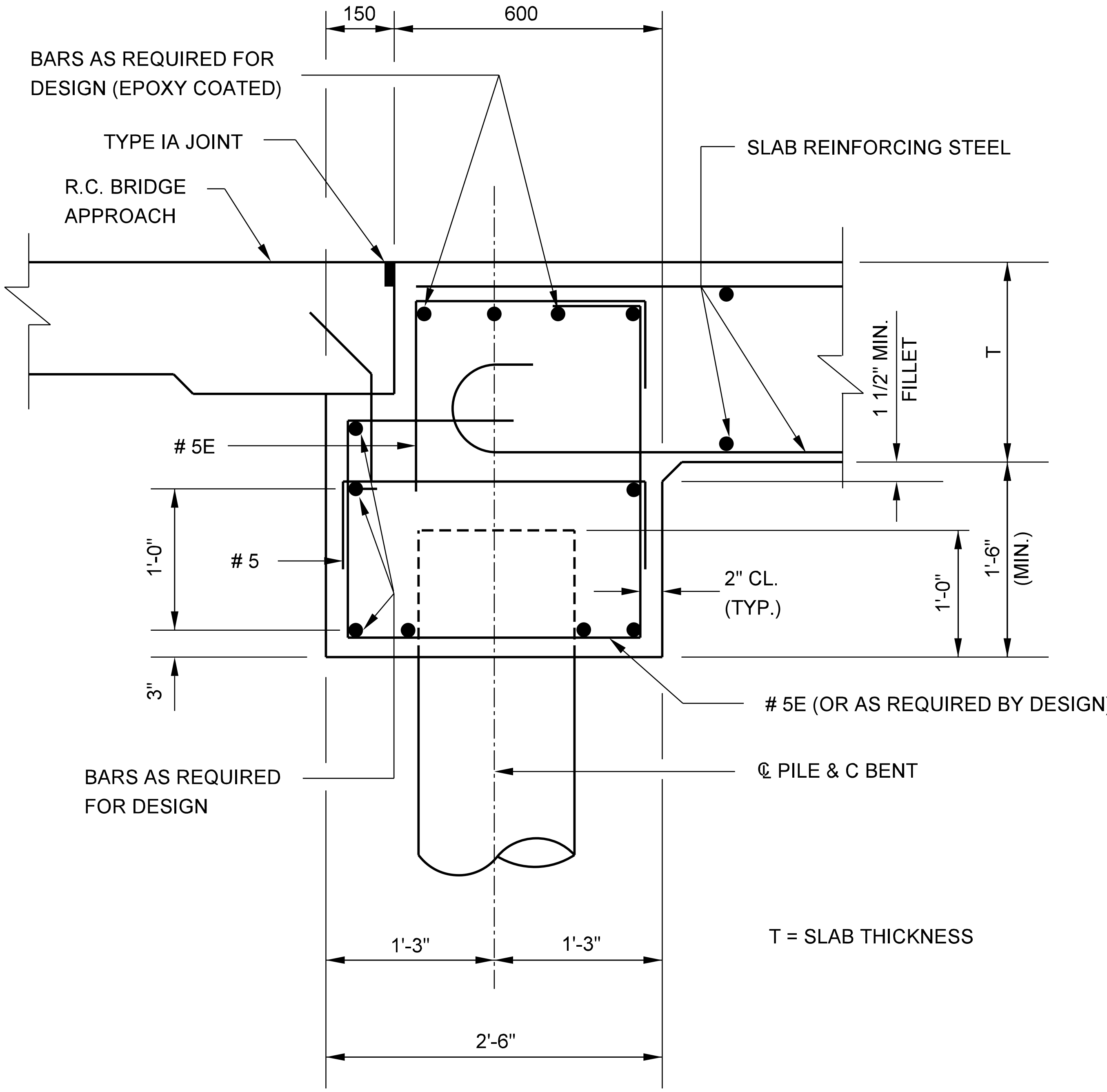


1. LONGITUDINAL BARS AS REQUIRED FOR DESIGN.
2. SHRINKAGE AND TEMPERATURE REINFORCEMENT (SEE FIGURE 62-3D)
3. DISTRIBUTION REINFORCEMENT IN ACCORDANCE WITH LRFD ARTICLE 5.14.4.1.
4. SUPPORT BARS (MAX. SPACING 3'-0")

LONGITUDINAL STEEL BAR SIZE	SUPPORT BAR SIZE
# 6 OR 7	# 5
# 8	# 6
# 9	# 6
# 10 OR LARGER	# 7

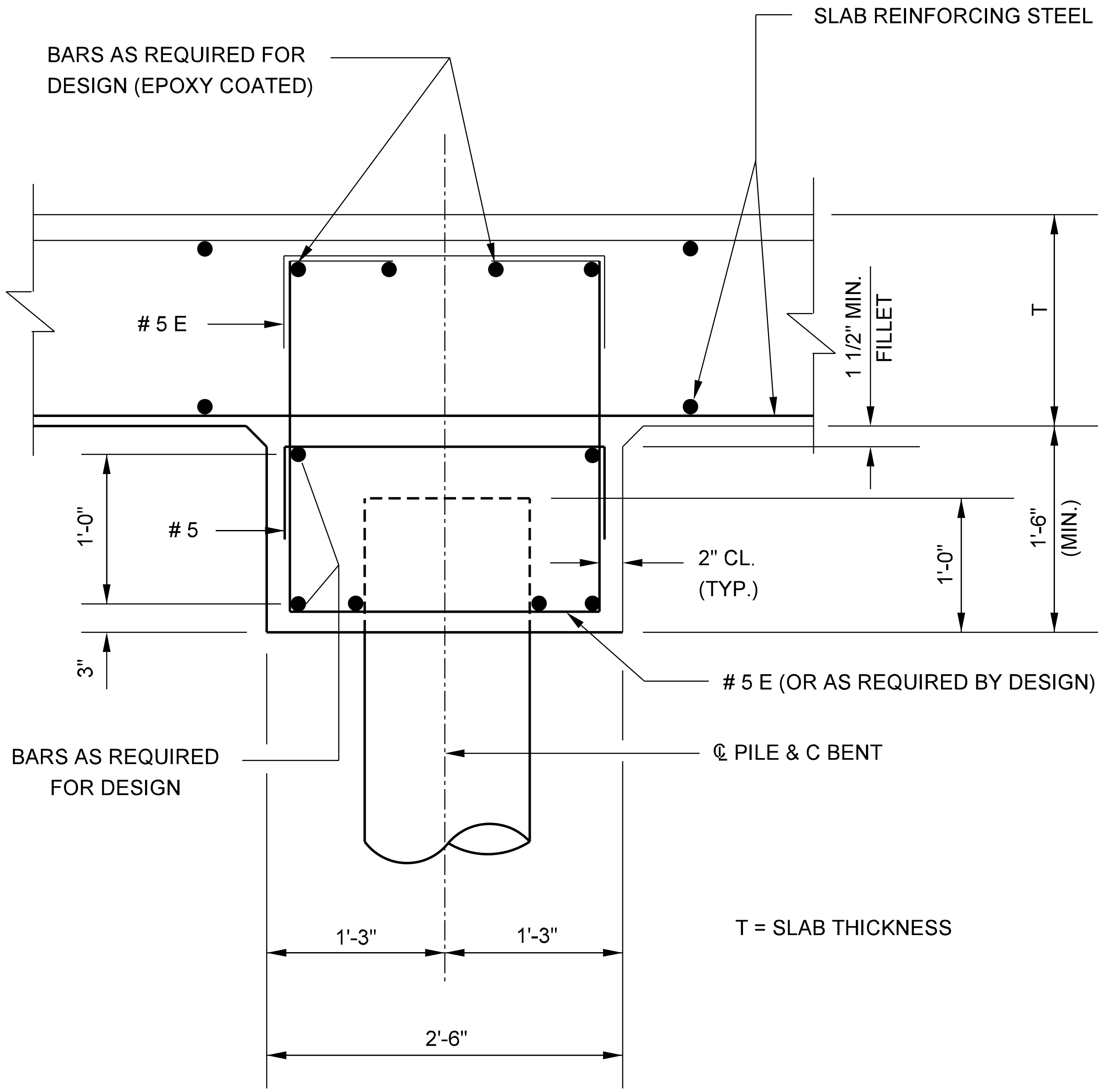
INTEGRAL CAPS AT SLAB SUPERSTRUCTURE
(Half Longitudinal Section)

Figure 62-3F



INTEGRAL CAPS AT SLAB SUPERSTRUCTURE
(Section Through End Bent)

Figure 62-3G



INTEGRAL CAPS AT SLAB SUPERSTRUCTURE
(Section Through Interior Bent)

Figure 62-3H