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Chapter Sixty-three

PRESTRESSED CONCRETE

63-1.0 INTRODUCTION

Prestressed concrete is probably the most important structural discovery of the 20th century. Its widespread acceptance testifies to its economy, reliable structural resistance, ductility and durability. It successfully combines the compressive resistance of concrete with the tensile resistance of high-strength steel. It permits a free manipulation of concrete stresses during and after construction and crack control in both flexural and shear design of beams.

For highway bridge construction, the primary use of prestressed concrete is in precast concrete beams and segmentally built superstructures, although prestressing of decks and substructures may become more common.

63-2.0 GENERAL

The generic word "prestressing" relates to a method of construction in which a steel element is stretched and anchored to the concrete. Upon release of the stretching force, the concrete will largely be under permanent compression and the steel under permanent tension. There are two methods of applying the prestressing force, as discussed in the following sections.

63-2.01 Pretensioning

In this method, stressing of the steel strands is done before the concrete is placed. It is practical only with factory- or mass-production facilities, because permanent, external anchorages are required to resist the reaction of the stressed strands. Once the concrete surrounding the steel attains a specified minimum strength, the strands are cut by which the prestressing force is transmitted to the concrete by bond-and-wedge action at the beam ends. The initial prestress is immediately reduced due to the elastic shortening of the concrete. Further losses will occur due to shrinkage and creep of concrete and relaxation of prestressing steel.

The generic word "prestress" is often used to mean "pretensioning" as opposed to "post-tensioning."

63-2.02 Post-Tensioning

In this method, tensioning of the steel is accomplished after the concrete has attained a specified minimum strength. The strands, usually forming tendons, are pulled or pushed into ducts cast into the concrete. Upon attaining the specified prestressing level, the tendons are anchored to the concrete and the jacks are released. Several post-tensioning systems and anchorages are used in the United States. The best information may be directly obtained from the manufacturers. Post-tensioned concrete is also susceptible to shrinkage and creep, although at a reduced magnitude because a significant portion of shrinkage usually occurs by the time of stressing, and the rate of creep decreases with the age at which the prestress is applied. Upon completion of stressing, the ducts are pressure filled with grout, which protects the tendons against corrosion and assures composite action with the beam by bond. Post-tensioning can be applied in phases to further increase the load-carrying capacity and better match the phased dead loads being applied to the beam.

In the United States, where industries are more inclined toward methods of mass production, pretensioning is more popular. Presently, however, it would appear that flexibility and economy can justify the sophistication required in the design and construction of post-tensioned concrete structures.

63-2.03 Partial Prestressing

In this hybrid design, both mild reinforcement and prestressing strands are present in the tension zone of a beam. The idea of partial prestressing, at least to some extent, originated from a number of research projects which indicated fatigue problems in prestressed beams. Fatigue is a function of the stress range in the strands, which may be reduced by placing mild steel parallel to the strands in the cracked tensile zone to share live-load induced stresses. In these projects, based on a traditional model, however, the fatigue load was seriously overestimated. The correct fatigue load provided by the *LRFD Specifications*, and discussed in Chapter Sixty-two, is a single design vehicle with reduced weight which is not likely to cause fatigue problems unless the beam is grossly under-reinforced. The two problems relative to partial prestressing are as follows:

1. A partially prestressed design usually results in more tension in the beam at service loads.

2. Tools for accurate analysis are not readily available to accurately predict stress-strain levels of different steels in the cross section.

For these reasons, partial prestressing is not permitted.

63-3.0 BASIC CRITERIA

63-3.01 Concrete

The following will apply to concrete.

1. The allowable design compressive strength of normal weight concrete at 28 days, f'_c, should be in the range as follows:

   a. prestressed box beam: 34 MPa to 48 MPa
   b. prestressed I-beam: 34 MPa to 48 MPa
   c. prestressed bulb-tee beam: 41 MPa to 55 MPa

   However, specifying a design strength higher than 45 MPa for a box beam or an AASHTO I-beam, and 48 MPa for a bulb-tee beam simply to make further refinements to the strand pattern is, generally, not cost effective and is not recommended.

   Exceptions to the above limits will be allowed for higher strengths if the designer can document that a higher strength is of significant benefit to the project, that the higher strength can be effectively produced, and approval is obtained from the Design Division Chief.

2. At release of the prestressing force, the compressive strength of concrete should not be less than 28 MPa and should not exceed 7 MPa less than the specified 28-day strength. The specified concrete compressive strength at release should be rounded to the next highest 1.0 MPa.

3. The modulus of elasticity of concrete based on normal-density concrete of 2320 kg/m^3 may be taken as $4800\sqrt{f'_c}$.

4. An ultimate strain of concrete in compression of 0.003 mm/mm should be used.

5. The maximum aggregate size should be limited to 19 mm.

63-3.02 Prestressing Strands

Prestressing strands should be of low-relaxation type with a minimum tensile strength of 1860 MPa (270 ksi). Unless there is a compelling reason to do otherwise, only the following three-strand diameters should be used.

1. Nominal 9.53 mm, A_s = 54.84 mm^2 (0.085 in^2) (e.g., in stay-in-place deck panels).

2. Nominal 12.70 mm, A_s = 107.74 mm^2 (0.167 in^2) (e.g., in beams and post-tensioned structures).

3. Nominal 15.24 mm, A_s = 140.0 mm^2 (0.217 in^2) (e.g., in bulb-tee beams and post-tensioned structures).

The steel should satisfy the requirements of AASHTO M203M (ASTM A416) and its supplement. Figure 63-3A illustrates a typical stress-strain diagram for these strands. The curve can best be approximated as follows:

1. a straight elastic line corresponding to $E_p \in_p$, where the modulus of elasticity E_p = 197 000 MPa (28 500 ksi);

2. a curved transition section, which is rather small for a low-relaxation strand but large for a stress-relieved strand;

3. a straight, strain-hardening line corresponding to $1600 + 8275 \in_p$; and

4. a plateau at f_{pu} = 1860 MPa (270 ksi).

The plateau is attained only at about $\in_p = 0.0317$, and the guaranteed fracture strain limit is $\in_p = 0.0400$. For low-relaxation strands, the transition curve could be neglected in the computations. Yield strength is usually defined as the stress at $\in_p = 0.0100$, which should be 0.90 f_{pu} = 1674 MPa for low-relaxation strands and 0.85 f_{pu} = 1581 MPa for stress-relieved strands. For either steel, the plateau can only be attained with an under-reinforced section.

See Table 5.9.3-1 of the *LRFD Specifications* for stress limits for prestressing tendons (strands). Strands should have minimum center-to-center distances as shown in Table 5-10.3.3.1-1 of the *LRFD Specifications*. Top strands in a box beam should be placed near the sides of the beam.

Consider placing at least two strands in the top flange of an I-beam. This will significantly reduce the need for debonded strands and will facilitate support of the mild reinforcement cage. The plans should include a note indicating if these strands are to be cut at the centerline of beam. Draped strands should only be considered in a bulb-tee beam when required by the design. The maximum allowable compressive strengths, tensile strengths, strand debonding and top strands should be considered when evaluating the need for draped strands. If draped strands are used, the maximum allowable hold-down force per strand shall be 17 kN (3.8 kips), with a maximum total hold-down force of 170 kN (38 kips).

Prestressing threadbars should have a minimum tensile strength of either 1030 MPa (150 ksi) or 1100 MPa (160 ksi). The diameters of the bars typically range from 26 mm to 36 mm. The steel should satisfy the requirements of AASHTO M275 M (ASTM A 722). They are primarily used for grouted construction. If the bars are used for permanent non-grouted construction, the bars shall be epoxy coated. Most bars are available in lengths up to 18 m. If couplers are used to connect bars for lengths of longer than 18 m, they should be enclosed in duct housings long enough to permit the necessary movement. The yield strength should not exceed 80% of the tensile strength.

See Table 5.9.3-1 of the *LRFD Specifications* for stress limits for deformed high-strength bars.

63-3.03 Ducts and Anchorages

In post-tensioned construction, the free passage of the tendons is assured by casting tendon ducts into the concrete. Ducts are usually rigid or semi-rigid, galvanized steel or polyethylene. *LRFD Specifications* Article 5.4.6.1 recommends polyethylene for corrosive environments, such as in a bridge deck or in substructure elements under joints. The contract documents shall indicate the type of duct material to be used.

Ducts for a post-tensioned bulb-tee beam shall be round, semi-rigid, galvanized-metal ducts. The wall thickness shall not be less than 28 gage. Prebending of ducts will be required for duct radii less than 900 mm and should be shown on the plans. Radii that require prebending should be avoided if possible. The minimum radius of ducts shall not be less than 6000 mm except in anchorage zones where 3600 mm will be permitted. The radius of polyethylene ducts shall not be less than 9000 mm.

If the bridge is constructed with post-tensioning precast components together longitudinally and/or transversely by use of a cast-in-place concrete joint, the end of the duct should be extended beyond the concrete interface for not less than 75 mm and not more than 150 mm to facilitate joining the ducts. If necessary, the extension could be in a local blockout at the concrete interface. Joints between sections of ducts shall be positive metallic connections, which do not result in angle changes at the joints. Waterproof tape shall be used at all connections.

Show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding 1.5 m. Where the rate of curvature of the duct exceeds one-half degree per meter, offsets shall be shown at intervals not exceeding 750 mm. In a region of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Curved ducts that run parallel to each other or around a void or re-entrant corner shall be sufficiently encased in concrete and reinforced as necessary to avoid radial failure (pull-out into the other duct or void).

If the precast beam is stored for a long period of time or if the tendon is to be stored or grouted in freezing weather, a drain hole should be provided at the low points in the duct profile.

For a multiple-strand tendon, the outside diameter of the duct shall not be more than 40% of the least gross concrete thickness at the location of the duct. The internal free area of the duct shall be at least 2.5 times the net area of the prestressing steel. See Article 5.4.6.2 of the *LRFD Specifications*. Upon completion of post-tensioning, the ducts must be grouted. The strength of the grout should be comparable to that of the beam concrete.

All ducts and anchorage assemblies for post-tensioning shall be provided with pipes or other suitable connections at each end for the injection of grout after prestressing. Ducts over 60 m in length shall be vented at all high points of the tendon profile. Vents shall be 12-mm minimum diameter standard pipe or suitable plastic pipe. All connections to the ducts shall be made with metallic or plastic fasteners. Waterproof tape shall be used at all connections to vent and grouting pipes.

Plastic components, if selected and approved, shall not react with the concrete or enhance corrosion of the prestressing steel and shall be free of water-soluable chlorides. The vents shall be mortar tight, taped as necessary, and shall provide means for injection of grout through the vents and for positive sealing of the vents. Ends of steel vents should be removed at least 25 mm below the concrete deck surface (if appropriate) after the grout has set. Ends of plastic vents should be removed to the surface of the concrete after the grout has set. Grout injection pipes should be fitted with positive mechanical shut-off valves. Vents and ejection pipes should be fitted with valves. Caps should not be removed or opened until the grout has set.

Although allowed in the *LRFD Specifications*, bundling of ducts will not be permitted. The clear distance between adjacent ducts should not be less than 38 mm or 1.33 times the maximum size of aggregate in any direction.

If the distance between anchorages exceeds 100 m, jacking at both ends should be considered. One or two end stressings are to be determined by the design and shown on the plans.

Figure 63-3B shows a typical tendon trajectory for a continuous beam end span. The geometry is composed from radius and straight segments as opposed to a parabolic trajectory, which is sometimes used. The use of simple radii and straight segments is preferred over parabolic trajectories because it is simpler to layout and has better structural efficiency due to the increased freedom of its geometry.

There is a great variety of commercially available anchorages. They normally consist of steel blocks with holes or slots in which the strands are individually anchored by friction with the help of wedges. In the vicinity of the anchor block or coupler, the strands are fanned out to accommodate the space requirement. The fanned-out part of the tendon is housed in a transition shield, often called a trumpet, which could be either steel or polyethylene, regardless of the material for the duct proper. A trumpet with a smooth, tangential transition to the ducts should be used.

Values of the wobble and curvature friction coefficients and the anchor set loss assumed for the design shall be shown on the plans.

<u>63-3.04 Loss of Prestress</u>

Loss of prestress is defined as the difference between the initial stress in the strands (just after seating of strands in the anchorage) and the effective prestress in the member (at a time when concrete stresses are to be calculated). This definition of loss of prestress includes both instantaneous losses and losses that are time dependent.

For a pretensioned member, prestress losses due to elastic shortening, shrinkage, creep of concrete, and relaxation of steel must be considered. This is reflected in Equation 5.9.5.1-1 of the *LRFD Specifications*.

For a post-tensioning application, friction between the tendon and the duct and anchorage seating losses during the post-tensioning operation must be considered in addition to the losses considered for a pretensioned member. This is reflected in Equation 5.9.5.1-2 of the *LRFD Specifications*.

Some of the important variables affecting loss of prestress are the concrete modulus of elasticity and creep and shrinkage properties. These variables can be somewhat unpredictable for a given concrete mixture and its placement procedure. These conditions are not fully controlled by the designer. Therefore, the estimation of losses should not be overemphasized at the expense of other more important issues during the design process.

Prediction of prestress losses may be determined by means of the approximate lump-sum estimate method, the refined itemized estimate method, or a detailed time-dependent analysis.

The *LRFD Specifications* provides guidance for the first two methods. The refined itemized estimate method should be used for the final design of a nonsegmental prestressed concrete member. For a post-tensioned concrete member with multistage construction and/or prestressing, the prestress losses should be computed by means of the time-dependent analysis method. The approximate lump-sum estimate method may be used for preliminary design only.

Examples for determining prestress losses can be found in of *Design of Highway Bridges Based on AASHTO LRFD Bridge Design Specifications*, Chapter Seven, published by John Wiley & Sons, Inc., 1997, and the *PCI Bridge Design Manual,* Chapter Eight, first published in 1997.

63-3.04(01) Elastic Shortening

Once the strands at the ends of a pretensioned member are cut, the prestress force is transferred to and produces compression in the concrete. The compressive force on the concrete causes the member to shorten with an accompanying loss of prestress.

The loss in prestress due to elastic shortening in a pretensioned member shall be computed by means of Equation 5.9.5.2.3a-1 of the *LRFD Specifications*. The following modulus of elasticity values should be used.

1. E_p = 197,000 MPa, the modulus of elasticity of the prestressing steel (LRFD Article 5.4.4.2).

2. $E_{ci} = 4800\sqrt{f'_c}$, the modulus of elasticity of concrete at transfer of the prestressing force (LRFD Eq. C5.4.2.4-1).

If the centroid of the prestressing force is below the centroid of the concrete member, the member will be lifted upward at transfer, and the self-weight of the member will be activated. The concrete stress at the centroid of the prestressing tendons is then given by the following equation.

$$f_{cgp} = \frac{P_i}{A_c} + \frac{(P_i e)e}{I_c} - \frac{M_b e}{I_c}$$

where:

P_i = prestressing force at transfer
A_c = area of the concrete beam
I_c = moment of inertia of concrete beam
e = eccentricity of prestressing steel at midspan
M_b = moment at midspan due to self-weight of beam

The force P_i will be slightly less than the transfer force because these stresses will be reduced by the elastic shortening of the concrete and the relaxation of tendons between the time of jacking and transfer.

Recognizing that this would require the use of a trial-and-error process of design iterations, Article 5.9.5.2.3a allows P_i to be based on a prestressing tendon stress of 0.70 f_{pu} for low-relaxation strands.

Strands that are placed in the top flange of the beam for the purpose of reducing the tensile stresses may be neglected for the determination of prestress losses due to elastic shortening.

As an alternative to the above method of calculation, Section 8.6.7.1 of the *PCI Bridge Design Handbook* provides for an alternative method of calculating elastic shortening losses.

For a post-tensioned member, there will be no loss of prestress due to elastic shortening if all the tendons are tensioned simultaneously. No loss occurs because the post-tensioning force compensates for the elastic shortening as the jacking operation progresses. If the tendons are tensioned sequentially, the first tendon anchored will experience a loss due to elastic shortening equal to that specified above for a pretensioned member.

Each subseqent tendon that is post-tensioned will see a fraction of the pretensioned loss, with the last tendon anchored having no loss. The average post-tensioned loss would be one-half of the pretensioned loss if the last tendon also had a loss. Because the last tendon does not have a loss, the loss of prestress due to elastic shortening for a post-tensioned member is given by Equation 5.9.5.2.3b-1 of the *LRFD Specifications*.

63-3.04(02) Shrinkage

Shrinkage of concrete is a time-dependent loss of prestress that is influenced by the curing method used, the volume-to-surface ratio of the member, the water/cement ratio of the concrete mix, and the ambient relative humidity, H.

The *LRFD Specifications* provides expressions for prestress loss due to shrinkage that are a function of average ambient relative humidity, H, and are given as Equations 5.9.5.4.2-1 and 5.9.5.4.2-2 for a pretensioned and a post-tensioned member, respectively. The average ambient relative humidity may be taken as 70%, which results in a shrinkage loss of 45.0 MPa for a pretensioned member or 33.5 MPa for a post-tensioned member.

For a post-tensioned member, the shrinkage loss in the tendons will be less than that for a pretensioned member because the concrete has additional drying time before the prestress is applied.

63-3.04(03)Creep

Creep of concrete is a time-dependant phenomenon in which deformation increases under constant stress due primarily to viscous flow of the hydrated cement paste. Creep depends on the age of the concrete, the type of cement, the hardness of the aggregate, the proportions of the concrete mixture, and the method of curing. The additional long-time concrete strains due to creep can be more than double the initial strain at the time load is applied.

The expression for prestress loss due to creep is a function of the concrete stress at the centroid of the prestressing steel at transfer, f_{cgp}, and the change in concrete stress at the centroid of the prestressing steel due to all permanent loads except those present at transfer, Δf_{cdp}, is given as Equation 5.9.5.4.3-1 of the *LRFD Specifications*. Equation 5.9.5.4.3-1 utilizes the same value of f_{cdp} as defined and discussed in Article 5.9.5.2.3 on elastic shortening. The value of Δf_{cdp} is computed by applying the deck weight and the weight of any interior diaphragms to the non-composite section and the composite dead loads to the composite section. Any wearing surface, which will be applied during the initial construction, should be included in the composite dead loads. However, a future wearing surface shall not be included. The values of f_{cgp} and Δf_{cdp} should be calculated at the point of maximum moment.

This prestress loss due to creep can be used for any prestressed concrete member. The signs on the stresses are based on the usual situation where the tendon eccentricity, e, is below the center of gravity of the section and opposing the dead load moments. Strands that are placed in the top flange of the beam for the purpose of reducing the tensile stresses may be neglected for the determination of prestress losses due to creep.

63-3.04(04) Relaxation

Relaxation of the prestressing tendons is a time-dependent loss of prestress that occurs when the tendon is held at constant strain. The total relaxation loss Δf_{pR} is separated into the two components as follows:

$$\Delta f_{pR} = \Delta f_{pR1} + \Delta f_{pR2}$$

where Δf_{pR1} is the relaxation loss at transfer of the prestressing force and Δf_{pR2} is the relaxation loss after transfer.

The prestress loss due to relaxation at transfer, Δf_{pR1}, for a pretensioned member shall be computed from Equations 5.9.5.4.4b-1 and 5.9.5.4.4b-2 of the *LRFD Specifications* for stress-relieved and low-relaxation strands, respectively.

The initial jacking stress, f_{pj}, is typically $0.75f_{pu}$ for low-relaxation strands or $0.70f_{pu}$ for stress-relieved strands (Table 5.9.3-1 of the *LRFD Specifications*). The yield strength of prestressing steel, f_{py}, for stress-relieved and low-relaxation strands may be taken from Table 5.4.4.1-1 of the

Specifications. For both stress-relieved and low-relaxation strands, the strand tensile strength, f_{pu}, is 1860 MPa. Also, the time, t, between anchoring of the stressed strands and the transfer of prestress to the member may be taken as 18 hours (0.75 day) for pretensioned beams of usual design.

Determining and substituting the values for f_{pj}, f_{py} and t into Equations 1 and 2 yields the following:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{10}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} = \frac{\log(24.0 \text{ x } 0.75)}{10}\left[\frac{1302}{1581} - 0.55\right]1302 = 44.7 \text{ MPa}$$

for stress-relieved strands, and

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} = \frac{\log(24.0 \text{ x } 0.75)}{40}\left[\frac{1395}{1674} - 0.55\right]1395 = 12.4 \text{ MPa}$$

for low-relaxation strands.

The relaxation loss, Δf_{pR1} , which is typically small, should be added to the computed elastic shortening loss, Δf_{pES}, when determining the initial prestress loss used to check beam stresses at transfer.

The commentary in the *LRFD Specifications* states that those relaxation losses prior to transfer are accounted for during fabrication of a prestressed member. However, this is not standard practice. The only adjustments to the prestressing force that are commonly made by producers are those required for temperature compensation, bed or form deformation, and chuck seating.

The prestress loss due to relaxation after transfer, Δf_{pR2}, for stress-relieved strands is a basic value of 138 MPa, which is reduced continually with time as the other prestress losses reduce the tendon stress. The elastic shortening loss, Δf_{pES}, occurs almost instantaneously so that its effect is largest. The losses due to shrinkage, Δf_{pSR}, and creep, Δf_{pCR}, take place over a period of time and have a smaller effect. The losses due to friction, Δf_{pF}, for a post-tensioned beam only, are somewhere between the two. The loss in prestress due to relaxation after transfer for stress-relieved strands shall be computed from Equations 5.9.5.4.4c-1 and 5.9.5.4.4c-2 for a pretensioned and a post-tensioned member, respectively. For low-relaxation strands, use 30% of Δf_{pR2} given by Equation 1 or 2.

63-3.04(05) Anchorage Set

In post-tensioned construction, not all of the stress developed by the jacking force is transferred to the member because the tendons slip slightly as the wedges, plates, shims, etc., seat themselves in the anchorage. The anchorage slip or set Δ_A is assumed to produce an average strain over the length of a tendon L, which results in an anchorage set loss as follows:

$$\Delta f_{pa} = \frac{\Delta_A E_p}{L}$$

where E_p is the modulus of elasticity of the prestressing tendon. Anchorage slip or set in a post-tensioning system is generally specific to a particular stressing system and is a given distance. The range of Δ_A varies from 3 to 10 mm with a value of 6 mm often assumed for strands. Bar tendons may have sets as low as 2 mm. For long tendons, the anchorage set loss is relatively small, but for short tendons it could become significant. It may often be possible to increase the initial prestress to compensate for the anchorage slip or set.

63-3.04(06) Friction

For a pretensioned member with draped strands, friction losses will occur at the hold-down points and should be accounted for by the fabricator in the stressing yard.

Because ducts and sheaths are used for post-tensioning, friction occurs where the cable makes contact with the duct. Contact will occur due to deliberate curvature of the duct, which is called the *curvature effect* and also due to unintended wobbling of the duct, which is called the *wobbling effect.* The coefficient of friction, μ, between the tendon and duct is an important quantity, as is the angle change of the cable. Design values for μ (coefficient of friction) and K (wobble friction coefficient) can be found in *LRFD Specifications* Table 5.9.5.2.2b-1. Some of these coefficients can vary significantly. The characteristics of the post-tensioning system that is to be used should be known so as to accurately estimate friction losses.

Losses due to friction between the internal tendons and duct wall shall be per Article 5.9.5.2.2b-1. Anchorage set causes a reversal of the friction effect near the end of the member; i.e., the tendon is sliding the opposite way. Therefore, the cable force will increase from the end of the member to a point where the anchorage set of the cable is achieved. Where large discrepancies occur between measured and calculated tendon elongations, in-place friction tests should be performed.

63-3.05 Strand Transfer and Development Lengths

The transfer length is the length of strand over which the prestress force is transferred to the concrete by means of bond and friction. The transfer length is generally very small and is usually in the range of 500 mm to 1000 mm from the end of the prestressed member. The *LRFD Specifications* indicates that the transfer length may be assumed to be 60 strand diameters. The stress in the strand is assumed to vary linearly from zero at the end of the member, or the point where the strand is bonded if debonding is used, to the full effective prestress force at the end of the transfer length.

The development length is the length of strand required to develop the stress in the strand corresponding to the full flexural strength of the member. Strand development length is the length required for bond to develop the strand tension at ultimate flexure. The transfer length is included as part of the development length. Equation 5.11.4.2-1 of the *Specifications* calculates the required development length l_d. Prestressing strands shall be considered fully bonded beyond the critical section for development length. The development length for debonded (shielded) strands shall be in accordance with Article 5.11.4.3.

Where debonded (shielded) strands are used, the following guidelines should apply.

1. In a bulb-tee beam, not more than 25% of the total number of strands and no more than 40% in each horizontal row shall be debonded. The allowable percentage of debonded strands for an AASHTO I-beam or a box-beam shall be no more than 50% of the total number of strands and of the strands in each horizontal row. Strands placed in the top flange of the beam are not to be included in the above percentages.

2. Exterior strands in each horizontal row should not be debonded.

3. Bonded and debonded strands should preferably alternate both vertically and horizontally.

4. Debonding termination points should be staggered at intervals of not less than 1m
.

5. Not more than four strands, or 40% of the total debonded strands, whichever is greater, should be terminated at any one point.

6. See Article 5.11.4 of the *Specifications* for additional guidelines.

7. Consider placing 2 strands in the top of a box beam, 2 or 4 strands in the top flange of an I-beam, or up to 6 strands in a bulb-tee beam. This may significantly reduce the need for debonded strands in the bottom of the beam. Where strands are placed in the top flange, a note should be shown on the plans indicating that these strands are to be cut at the center of the beam after the bottom strands are released and the pocket is then to be filled with grout. The top strand may sometimes not need to be cut if ultimate moment controls the number of strands in the bottom flange.

8. Top strands in a concrete box beam should be placed near the sides of the box.

63-4.0 PRESTRESSED BEAM SECTIONS

63-4.01 General

The type of beams used in the superstructure should be selected based upon economy and appearance. The following standard prestressed concrete beam sections are used.

1. AASHTO I-beams types I through IV;
2. Indiana bulb-tee beams; and
3. Indiana composite and non-composite box beams.

To ensure that the structural system has an adequate level of redundancy, a minimum of four beam lines should be used on an INDOT-route structure. Three beam lines may be used on a local-public-agency-route structure if the designer obtains written approval from the appropriate LPA elected official. Section 61-5.02 provides width criteria for deck overhangs.

An alternative prestressed concrete beam section may be considered if the designer can justify its use. The use of beam sections not available through local producers will usually be more expensive if the forms must be purchased or rented for a small number of beams. One or more beam fabricators should be contacted early in project development to determine the most practical and cost-effective alternative beam section for a specific site.

63-4.02 AASHTO I-Beams Types I Through IV

See Section 63-13.0 for details and section properties of these beams. The I-beam type IV is not generally used unless widening of an existing bridge is required. The 1372-mm deep bulb-tee beam will typically be used for a new structure where this particular member depth and span length is required. See Section 59-3.02(05) for additional information on AASHTO I-beams.

63-4.03 Indiana Bulb-Tee Beams

See Section 63-14.0 for details and section properties of these beams. For a long-span bridge, bulb-tee beams with a 1524 mm wide top flange should be considered for improved stability during handling and transporting. Draped strands may be considered for use in a bulb-tee beam, but should only be considered when tensile stresses in the top of the beam near its end are exceeded using straight strands. For additional information on draped strands, see Section 63-5.0. Semi-lightweight concrete may be used for this type of beam if it is economically justified. See Section 63-10.0. For additional information on bulb-tee beams, see Section 59-3.02(06).

63-4.04 Indiana Composite and Non-Composite Box Beams

See Section 63-15.0 for details and section properties of these beams. It is not acceptable to use non-composite box beams for a permanent State highway bridge. The use of the non-composite box beam is limited to a non-Federal-aid local public agency bridge or a temporary bridge. The desirable limit for the end skew is 30 degrees. An end skew of over 30 degrees should be avoided unless measures have been considered for potential warping or cracking of the beam at its ends and congested reinforcement in the acute angle corner of the beam.

For a spread box beam structure, 200-mm thick diaphragms shall be placed within the box section for increased stability and torsion resistance during delivery and erection of the beams. The maximum spacing of these diaphragms shall be 7.6 m.

For an adjacent box beams structure, interior diaphragms shall be provided to accommodate the transverse tension rods or tendons. Effective means for transferring shear between the box beams should be provided (see Section 63-8.0). Because the longitudinal joints between adjacent box beams have shown a tendency to leak, use of prestressed adjacent box beams should be limited to where maintaining a thin construction depth is critical, where construction time is critical, or where substantial life-cycle cost savings can be clearly demonstrated.

Every void in a box beam should be equipped with a vertical drainage pipe to prevent accumulation of water and ice therein. The inside diameter of the pipe should be approximately 15 mm, and it should be located at the lowest point of the void in the finished structure.

If the cost of a superstructure using precast concrete AASHTO I-beams or bulb-tees is close to the cost of precast concrete spread box beams, the I-beam or bulb-tee superstructure is prefrred unless other factors such as a thin structure depth are critical.

See Section 59-3.02(05) for additional information on spread box beams. See Section 59-3.02(07) for additional information on adjacent box beams,.

63-5.0 STRAND CONFIGURATION AND MILD STEEL REINFORCEMENT

63-5.01 General

Proper detailing of strand configuration and mild reinforcing steel offers an opportunity to contribute to cost savings. Mild reinforcing steel should be detailed to allow its placement after the strands have been tensioned. If the reinforcement is a one-piece bar detailed around the strands, it requires that the strands be threaded through the closed bars. By using two-piece bars that can be placed after the strand is tensioned, the fabrication process is simplified.

When specifying concrete cover and spacing of strands and bars, the designer must consider reinforcing bar diameters and bend radii to avoid conflicts. As stated in Section 63-3.02, to support the reinforcing steel cage, many producers prefer to locate at least two strands in the top of each I-beam or bulb-tee below the top transverse bars and between the vertical legs of the web reinforcement.

63-5.02 Strand Configuration

See Sections 63-13.0, 63-14.0, and 63-15.0 for typical strand patterns for standard prestressed beam sections. Other strand patterns may be used if there is good reason for deviation from the standard pattern and as long as the AASHTO criteria for spacing and concrete cover are followed. If 11 strands are placed in a horizontal row in the bottom of a bulb-tee beam, the bending diagram for the vertical stirrup must be modified. The strand patterns shown may be used for both nominal 12.70 mm and 15.24 mm diameter strands. Section 63-3.02 provides criteria for the various strand diameters used.

The strand pattern configurations shown in Sections 63-13.0, 63-14.0, and 63-15.0 were developed in accordance with the following:

1. Minimum center-to-center spacing of prestressing strands should be 50 mm, instead of the 51 mm shown in Table 5.10.3.3.1-1 of the *LRFD Specifications*.

2. Minimum concrete cover for prestressing strands should be 40 mm, which includes the modification factor of 0.8 for a W/C ratio equal to or less than 0.40 (LRFD Article 5.12.3).

3. Minimum concrete cover to stirrups and confinement reinforcement should be 25 mm.

The strand pattern has been configured so as to maximize the number of vertical rows of strands that can be draped. Due to the relatively thin top flange of bulb-tee beams, strands placed in the top of the beam should be at least 150 mm from the outside edge of the flange.

63-5.03 Mild Steel Reinforcement

See Sections 63-13.0, 63-14.0, and 63-15.0 for typical mild steel reinforcement for the standard prestressed beam sections. The vertical shear reinforcement should be #13 stirrup bars where possible. To fully develop the bar for shear, the ends of the stirrup bar should include a standard 90-degree stirrup hook. The maximum spacing of the vertical stirrups shall be in accordance with Article 5.8.2.7 of the *LRFD Specifications*. The maximum longitudinal spacing of reinforcement for interface shear transfer shall be in accordance with LRFD Article 5.8.4.1.

A minimum of three horizontal U-shaped #13 bars should be placed in the web of each bulb-tee at the ends of the beam. See Section 63-14.0 for location and spacing of these bars. This reinforcement will help reduce the number and size of cracks, which may appear in the ends of the beams due to the prestress force. *LRFD Specifications* Article 5.10.10.1 also requires that vertical mild reinforcement be placed in the beam ends within a distance of one fifth of the member depth. This is to provide bursting resistance of the pretensioned anchorage zone. Enough mild reinforcing steel shall be provided to resist not less than 4% of the prestress force at transfer. The end vertical bars should be as close to the ends of the beam as possible. The stress in the reinforcing steel shall not exceed 140 MPa.

Confinement reinforcement (LRFD Article 5.10.10.2) shall be placed in the bottom flange of each I-beam or bulb-tee. The reinforcement shall be #10 bars spaced at 150 mm for a minimum distance of one and one-half the depth of the member from the end of the beam or to the end of the strand debonding, whichever is greater.

The minimum concrete cover for all tie and stirrup bars shall be 25 mm.

63-6.0 DESIGN OF PRESTRESSED CONCRETE BEAMS

63-6.01 General

This Section describes the general design theory and procedure for precast, prestressed (pre-tensioned) concrete beams. For specific design examples, see the *PCI Bridge Design Manual*, Chapter 9, or *Design of Highway Bridges Based on AASHTO LRFD Design Specifications*, by Barker and Puckett. The design of post-tensioned concrete beams will not be addressed in this Chapter.

A multi-span bridge using composite beams should be made continuous for live load if possible. The design of the beams for a continuous structure is approximately the same as that for simple spans except that, in the area of negative moments, the member is treated as an ordinary reinforced concrete section. The members shall be assumed to be fully continuous with a constant moment of inertia when determining both the positive and negative moments due to superimposed loads.

The load modifier η shall be as specified in LRFD Article 1.3.2 of the *Specifications*. The limit states (LRFD Article 3.4) to be used for design of the beams will usually consist of satisfying the requirements of Service I, Service III, and Strength I load combinations. Service III specifies a load factor of 0.80 to reduce the effect of live load at the service limit state. This combination is only applicable when checking allowable tensile stresses in the beams. Service I is used when checking compressive stresses only.

The resistance factor φ (LRFD Article 5.5.4) shall be as follows:

1. For flexure, 1.0. For design of the negative moment steel in the deck for a structure made continuous for composite loads only and having a poured-in-place continuity joint between the ends of the beams over the piers, 0.90.

2. For shear and torsion, 0.90 for normal weight concrete, 0.80 for semi-lightweight concrete, and 0.70 for lightweight concrete.

63-6.02 Lateral Stability

In addition to designing for the normal "in service" loading conditions, LRFD Article 5.5.4.3 of the *Specifications* indicates that buckling of precast members during handling, transportation, and erection shall be investigated. The INDOT *Standard Specifications* makes the contractor responsible for handling, storing, and erection of beams and other precast elements.

For a beam length of over 30 m, the designer should consult with a fabricator to determine whether such beams can be delivered to the project site.

63-6.03 Stage Loading

The loading conditions that affect the design of a prestressed beam are as follows:

1. The strands are tensioned in the bed prior to placement of the concrete. Seating losses, relaxation of the strands, and temperature changes affect the stress in the strands prior to placement of the concrete. It is the fabricator's responsibility to consider these factors during the fabrication of the beam and to make adjustments to the initial strand tension to make sure that the tension prior to release meets the design requirements.

2. The strands are released and the force is transferred to the concrete. After release, the beam will camber up and be supported at the beam ends only. Therefore, the region near the end of the member does not receive the benefit of bending stresses due to the dead load of the beam and may develop tensile stresses in the top of the beam large enough to crack the concrete. The critical sections for computing the critical temporary stresses in the top of the beam should be near the end and at all debonding points. If the designer chooses to consider the transfer length of the strands at the end of the beam and at the debonding points, the stress in the strands should be assumed to be zero at the end of the

beam or debonding point and should vary linearly to the full transfer of force to the concrete at the end of the strand transfer length.

The methods of relieving excessive tensile stresses near the ends of the beam are as follows:

a. debonding, wherein the strands are kept straight but wrapped in plastic over a predetermined distance;

b. adding additional strands in the top of the beam, debonding them in the middle third, and releasing them at the center of the beam; or

c. draping some of the strands to reduce the strand eccentricity at the end of the beam.

See Section 63-3.0 for additional criteria regarding draping and debonding of strands. The level of effective prestress immediately after release of the strands should include the effects of elastic shortening and the initial strand relaxation loss.

3. This condition occurs several weeks to several months after strand release. Camber growth and prestress losses are design factors at this stage. If a cast-in-place composite deck is placed, field adjustments to the haunch fillet thickness are usually needed to provide the proper vertical grade on the top of deck and to keep the deck thickness uniform. Reliable estimates of deflection and camber are needed to prevent excessive fillet thickness or to avoid significant encroachment of the top of beam into the bottom of the concrete deck. Stresses at this stage are usually not critical.

   Unless other more accurate methods of determining camber are utilized (see the *PCI Bridge Design Manual*, Section 8.7), the beam camber at the time of placement of the composite concrete deck shall be assumed to be the initial camber due to prestress minus the deflection due to the dead load of the beam times a multiplier of 1.75.

4. After an extended period of time, all prestress losses have occurred and loads are at their maximum. This is often referred to as the "maximum service load, minimum prestress" stage. The tensile stress in the bottom fibers of the beam at mid-span generally controls the design.

63-6.04 Flexure

63-6.04(01) General

Flexure design normally starts with the determination of the required prestressing level to satisfy service conditions. All load stages that may be critical during the life of the structure from the time prestressing is first applied should be considered. This is then followed by a strength check of the entire member under the influence of factored loads. The strength check seldom requires additional strands or other design changes. The weight of the future wearing surface, sidewalk (if not poured with the deck), and railings should be included in the design as composite dead loads. The weight of the railing and sidewalks may be distributed equally to all beams unless they are poured with the deck.

A 15-mm, non-structural concrete deck wearing surface should be deducted from the composite design. If the 20-mm haunch is considered in the composite section properties, its width should be transformed before it is used in the calculations. The additional dead load of the haunch, intermediate diaphragms, and optional metal deck forms (0.70 kN/m^2) should be included in the design of the beam.

For checking the allowable stresses in the beam, the following basic assumptions are made.

1. Plain sections remain plain, and strains vary linearly over the entire member depth. Therefore, composite members consisting of precast concrete beams and cast-in-place decks must be adequately connected so that this assumption is valid and all elements respond to superimposed loads as one unit.

2. Before cracking, stress is linearly proportional to strain.

3. After cracking, tension in the concrete is neglected.

63-6.04(02) Design Procedure

The tensile stresses at mid-span due to full dead and live loads plus effective prestress (after losses) usually controls the design as follows.

1. Compute the tensile stress due to beam self-weight plus any other non-composite loads such as deck, SIP forms, haunches, diaphragms, etc., applied to the beam section only.

2. Compute the tensile stress due to superimposed dead loads plus 80% of the live loads that are applied to the composite section.

3. Compute the net stress in the beam by subtracting the allowable tensile stress from the stresses computed in Steps 1 and 2 above. This will be the stress that needs to be offset by the prestressing. To find the prestressing required, solve the following equation for the effective prestress P_{se}.

$$f_b = \left(\frac{P_{se}}{A}\right) + \left(\frac{P_{se}e_c}{S_b}\right)$$

Where:

f_b = net stress in the beam

e_c = strand eccentricity

A = beam area

S_b = bottom fiber modulus

P_{se} The stimated number of strands = $\dfrac{P_{se}}{f_{pe}(area\ of\ one\ strand)}$

f_{pe} = effective prestress after losses, which may be approximated as 1170 MPa for 1860 MPa (270 ksi) low-relaxation strand.

4. Perform a detailed calculation of prestress losses (see Section 63-3.04) and repeat Step 3 if necessary.
5. Check stresses at the ends, strand debonding points (if applicable), drape points (if applicable), and mid-span at release and at service loads. Under normal load conditions, stresses at service loads will not govern at debonding points or drape points.

6. Check strength. Approximate formulas for bonded tendons for pretension steel stress for flanged or rectangular sections at ultimate flexure are shown in LRFD Article 5.7.3.1.1. The approximate formulas are based upon the following assumptions.

   a. The compression zone is either rectangular or T-shaped.

   b. The compression zone is within only one type of concrete for a composite member, and it is assumed to be within the deck concrete. Only fully tensioned strands near the tension face of the beam may be used. The top strands should be ignored.

   c. The effective pretension is not less than 50% of the ultimate strength of the strands.

   d. Steel content must be below the amount that causes the predicted steel stress to be lower than the yield strength.

   e. Because debonded strands require 25% more development length than bonded strands (LRFD Article 5.11.4), consideration should be given to development length beyond the point of debonding when computing the ultimate strength. The strands are adequately developed, or reductions accounted for, at all critical design sections such as in the maximum moment region.

The factored flexural resistance $M_r = \varphi M_n$, where φ = resistance factor of 1.00. It is then computed using the equations referenced in LRFD Article 5.7.3.2.3 for a rectangular section and LRFD Article 5.7.3.2.2 for a flanged section. If the beam section is other than a rectangular or flanged section, a general strain compatibility approach should be taken.

Maximum reinforcement should be checked in accordance with LRFD Article 5.7.3.3.1. 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 regions located within approximately the end one third of the beam or span, 1.33 times the factored moment will generally control.

Use LRFD Equation 5.7.3.3.2-1 to compute the cracking moment.

$$M_{cr} = S_c\,(f_r + f_{cpe}) - M_{dnc}\,(S_c/S_{nc} - 1) \leq S_c f_r$$

where:

M_{cr} = cracking moment (N-mm)

f_r = modulus of rupture of concrete as specified in Article 5.4.2.6 (MPa)

7. If necessary, revise the number of strands and repeat Steps 4 and 5.

Any loads placed on the bridge after the deck has hardened should be applied as composite loads if satisfactory bond is provided between the deck and the top of beam. This bond is usually accomplished by the use of a roughened concrete surface on the top of the beam and shear stirrups that project out of the top of the beam into the cast-in-place deck. Because the deck concrete is usually of a lower strength than the precast beam, the deck should be transformed into an equivalent beam by using the modular ratio n. The effective flange width must be determined in accordance with LRFD Article 4.6.2.6.1.

A fatigue check of the strands is generally not required unless the beam is designed to crack under service loads. Fatigue of concrete in compression is unlikely to occur in actual practice. Fatigue considerations for prestressed concrete components are addressed in LRFD Article 5.5.3.

63-6.05 Web Shear

63-6.05(01) Design Models

The AASHTO *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 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, and does not consider the specific details of how the force effects were introduced into the member. Therefore, only the sectional model approach will be discussed in this Section.

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, and *PCI Bridge Design Manual* Section 8.12 for more information regarding the strut-and-tie model.

Article 5.8.3 of the *Specifications* 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:

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

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

LRFD Eq. 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:

$$V_c = 0.83\,\beta \sqrt{f'_c}\; b_v d_v \qquad \text{(LRFD Eq. 5.8.3.3-3)}$$

The contribution of the web reinforcement is given by the following:

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

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.

Where the web shear reinforcement is vertical ($\alpha = 90^o$), V_s simplifies to the following:

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

Transverse shear reinforcement should be provided based on the following:

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

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

$$A_v = 0.83\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(LRFD Eq. 5.8.2.5-1)}$$

Where 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).

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 Table 5.8.3.4.2-1. For a section that does not meet the minimum transverse reinforcement requirements, Table 5.8.3.4.2-2 should be used to determine β and θ.

63-6.05(02) Design Procedure

The area and spacing of shear reinforcement must be determined at regular intervals (tenth points) along the span and at the critical section as described above. To design a member for shear, the factored shear should be determined due to applied loads at the section under consideration. The values for b_v and d_v should then be determined. The value of d_v (effective shear depth), which is the distance between the resultants of the tensile and compressive forces due to flexure, can be expressed as follows:

$$d_v = \frac{M_n}{\left(A_s f_y + A_{ps} f_{ps}\right)} \quad or \quad \left(d_e - \frac{a}{2}\right)$$

The value of d_v need not be less than the greater of $0.9d_e$ or $0.72h$.

The stress contribution from any draped strand, V_p, is then computed. V_p, the component of the effective prestressing force in the direction of the applied shear is the force per strand times the number of draped strands times $\sin\Psi$. The angle of drape, Ψ, as measured from the longitudinal axis of the beam.

Next, the factored shear stress is calculated using the following:

$$v = \frac{V_u - \Phi V_p}{\Phi b_v d_v} \qquad \text{(LRFD Eq. 5.8.2.9-1)}$$

The quantity v/f'_c is then computed, and a value of θ is assumed. For a prestressed member, a reasonable initial estimate for θ is 30°.

For a section that contains at least the minimum transverse reinforcement as specified in LRFD Article 5.8.2.5, the strain in the tensile reinforcement is calculated using the following:

$$\varepsilon_x = \left[\frac{(M_u/d_v) + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}\right] \le 0.001 \quad \text{(LRFD Eq. 5.8.3.4.2-1)}$$

If the section contains less than the minimum transverse reinforcement as specified in LRFD Article 5.8.2.5, use LRFD Eq. 5.8.3.4.2-2. If the value of ε_x from either Eq. 1 or 2 is negative, use LRFD Eq. 5.8.3.4.2-3.

The *Specifications* indicates that the area of prestressing steel, A_{ps}, must account for the lack of development near the ends of prestressed beams. Any mild reinforcement or strand in the compression zone of the member, which is taken as one-half of the overall depth (h/2), should be neglected when computing A_s and A_{ps} for use in this calculation. When evaluating a member with draped strands, near the ends of a typical beam, draped strands are near the top of the beam. Because of this, it is recommended that the straight and draped strands be considered separately in the analysis. The physical location of each strand is significant, and not the centroid of the group.

The variable f_{po} can be taken as the stress in the strands once the concrete is cast around them. The stress in the concrete is zero. Therefore, f_{po} can be conservatively taken as $0.7f_{pu}$ for the usual levels of prestressing. Within the transfer length of the strands at the end of the beam, f_{po} should be increased linearly from zero at the end of the beam to its full value at the end of the transfer length.

LRFD Table 5.8.3.4.2-1 is then entered for a section with minimum transverse reinforcement as specified in Article 5.8.2.5 with the values of v/f'_c and ε_x. The value of θ corresponding to v/f'_c and ε_x is compared to the assumed value of θ. If the values match, V_c is calculated using Eq.

5.8.3.3-3 with the value of β from the Table. If they do not match, the value of θ taken from the Table is used for another iteration. Of the quantities computed thus far, only ε_x will change with a new value of θ, so the effort required for additional iterations is minor. The same procedure is used for a section with less than the minimum transverse reinforcement, except that Table 5.8.3.4.3-2 is used.

After V_c has been computed, V_u must be checked to determine if it is greater than $0.5\Phi(V_c + V_p)$. If V_u exceeds this value, shear reinforcement is required. The quantity of shear steel is calculated using $A_v = (sV_s)/(f_y d_v \cot\theta)$. After determining the amount of shear reinforcement needed, the maximum spacing allowed by the *Specifications* should be checked as described in Article 5.8.2.7. The size of the shear stirrup should be a #13 bar where possible. Also, the amount of shear reinforcement should be checked to ensure that it is equal to or larger than the minimum value required by Eq. 5.8.2.5-1.

To ensure that the concrete in the web of the beam will not crush prior to yielding of the transverse reinforcement, the *LRFD Specifications* gives an upper limit of V_n, as follows:

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

$(V_c + V_s)$ should be less than or equal to $0.25f'_c\, b_v d_v$. Using the foregoing procedures, the transverse reinforcement can be determined at the tenth points along the beam and at the critical section.

In regions of high shear stress, such as near the support, the longitudinal (flexural) reinforcement must also be able to carry the additional stress due to shear; i.e., the horizontal component T of the diagonal compression field. Therefore, the amount and development of the longitudinal reinforcement must be greater than or equal to Eq. 5.8.3.5-1, as follows:

$$T = \frac{M_u}{d_v \Phi} + \frac{N_u \Phi}{2} + \left(\frac{V_u}{\Phi} - \frac{V_s}{2} - V_p\right)\cot\theta$$

This equation should be satisfied, especially near non-continuous supports where a substantial portion of the prestressing strands are either debonded or draped. Draped or debonded strands are not effective in contributing to this longitudinal reinforcement requirement because they are either above the midheight of the member (h/2), or they are not bonded to the concrete.

LRFD Article 5.8.3.5 requires that the longitudinal reinforcement on the flexural tension side of the member (below h/2) shall resist a tensile force of $(V_u/\varphi - 0.5V_s - V_p)\cot\theta$ at the inside edge of the bearing area at the simple end supports. The values of V_u, V_s, V_p, and θ, calculated for the critical section $0.5d_v\cot\theta$, or d_v from the face of the support, may be used. In calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the transfer length may be assumed.

63-6.06 Horizontal Interface Shear

A cast-in-place concrete deck designed to act compositely with precast concrete beams must be able to resist the horizontal shearing forces at the interface between the two elements. The following formula may be used to determine V_h.

$$V_h = V_u / d_e \qquad \text{(LRFD Eq. C5.8.4.1-1)}$$

The required strength should be less than or equal to the nominal strength and is as follows:

$$V_h A_{cv} \leq \Phi V_n$$

where $V_n = cA_{cv} + \mu[A_{vf}f_y + P_c]$. P_c, the permanent net compressive force normal to the shear plane, may be conservatively neglected.

LRFD Article 5.8.4.2 indicates that for concrete placed against clean hardened concrete with the surface intentionally roughened to an amplitude of 6 mm, the tops of the beams should be scored at 75-mm centers transverse to the top beam flange with a pointed tool to a depth of at least 6 mm.

c = 0.70 MPa
μ = 1.0 λ, where λ = 1.0 for normal density concrete

Therefore, for normal weight concrete cast against hardened, roughened, normal weight concrete, the above relationship may be reduced as follows:

$$V_h \leq \Phi\left[0.7 + \frac{A_{vf} f_y}{A_{cv}}\right]$$

where the minimum $A_{vf} \geq \dfrac{0.35 f_y}{A_{cv}}$.

The nominal shear resistance, V_n, used in the design shall satisfy the following
:

$$V_n \leq 0.2 f'_c A_{cv}\text{, or} \qquad \text{(LRFD Eq. 5.8.4.1-2)}$$

$$V_n \leq 5.5 A_{cv} \qquad \text{(LRFD Eq. 5.8.4.1-3)}$$

If the width of the interface surface is more than 1225 mm, a minimum of four #13 bars should be used for each row with two of the bars, one on each side of the flange, located near the outside edge of the flange. LRFD Article 5.8.4.1 requires that the maximum spacing of horizontal shear stirrups not exceed 600 mm. For a member such as a partial-depth deck panel, the minimum reinforcement requirement of A_{vf} may be waived if V_n / A_{cv} is less than 0.7 MPa.

63-6.07 Continuity for Superimposed Loads

The traditional method of making simply supported beams continuous is to construct a closure joint between the adjacent beam ends over the pier, conveniently as part of the diaphragm, and to place extra longitudinal steel in the deck over the pier support to resist the negative moment. Spans made continuous for live load are assumed to be treated as prestressed members in the positive moment zone between supports and as conventionally reinforced members in the negative moment zones over the support. The reinforcing steel in the deck should carry all of the tension in the composite section due to the negative moment. The longitudinal reinforcing steel in the deck that makes the girder continuous over an internal support shall be designed in accordance with LRFD Article 5.14.1.2.7b.

The compressive strength of the beam concrete should be used regardless of the strength of the cast-in-place concrete. Due to lateral restraint of the diaphragm concrete, ultimate negative moment compression failure in the PCA tests as described in *PCA Engineering Bulletin, Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders*, August 1969, always occurred in the girder, even though the diaphragm concrete strength was approximately 14 MPa less than that of the girder concrete. The negative moment reinforcement in the deck should therefore be designed using the compressive strength of the concrete in the precast elements. See Article 5.14.1.2.7 of the *LRFD Specifications*.

No allowable tension limit is imposed on the top fiber stresses of the beam in the negative moment region. However, crack width, fatigue, and ultimate strength should be checked. If partial-depth precast, prestressed concrete stay-in-place forms are to be used, such as for an AASHTO I-beam superstructure, only the top mat of longitudinal steel reinforcement should be used to meet the negative moment requirements.

63-6.08 Effect of Imposed Deformations

Potential positive moments at the piers should also be considered in the design of a precast, prestressed concrete beam structure made continuous for live load. Creep of the beams under the net effects of prestressing, self-weight, deck weight, and superimposed dead loads will tend to produce additional upward camber with time. Shrinkage of the deck concrete will tend to produce downward camber of the composite system with time. Loss of prestress due to creep, shrinkage, and relaxation will result in downward camber. Depending on the properties of the concrete materials and the age at which the beams are erected and subsequently made continuous, either positive or negative moments may occur over the continuous supports.

Where beams are made continuous at the relatively young age of less than 120 days from time of manufacture, it is more likely that positive moments will develop with time at the supports. These positive restraint moments are the result of the tendency of the beams to continue to camber upwards as a result of ongoing creep strains associated with the transfer of prestress. Shrinkage of the deck concrete, loss of prestress, and creep strains due to self-weight, deck weight, and superimposed dead loads all have a tendency to reduce this positive moment.

For a span of over 45 m in length or for concrete whose creep behavior is known to be poor, the designer may wish to consider making a time-dependent analysis to predict positive restraint moments at the piers. The *PCI Bridge Design Manual*, Section 8.13.4.3, describes two methods to evaluate restraint moments at the piers. If the designer has experience with similar spans and concrete creep properties, he or she may use the positive moment connection details at the piers that have proven successful in the past.

Unless positive-moment-connecting steel calculations are made, the minimum number of strands to be used for the positive moment connection over the pier shall be one-half of the strands (minimum 5 strands for a bulb-tee or I-beam type IV, 4 strands for I-beams types II or III, and 3 strands for I-beam type I) in the bottom row of the bottom flange of a bulb-tee or an I-beam.

The strands should be extended and bent up without the use of heat to make the positive moment connection. For a box beam, the minimum number of strands to be extended into the positive moment connection and bent up shall be 6 strands for a beam deeper than 686 mm or 4 strands for a beam equal to or less than 686 mm in depth.

The strands extended into the positive moment connection between beams shall not be debonded. The strands that are not used for the positive moment connection should be trimmed back to the beam end to permit ease of beam and concrete placement.

The prestressing strand and concrete strengths shall be as indicated in Section 63-3.0. The tensile and compressive stress limits shall be as shown in LRFD Article 5.9.4. The *LRFD Specifications* requires that only 80% of the live-load moment is to be applied when checking the tensile stress at service conditions.

63-7.0 DIAPHRAGMS

Reference: LRFD Article 5.13.2.2

63-7.01 General

A multi-girder bridge (except for one with adjacent box beams) shall have diaphragms provided at abutments, end bents, and interior piers or bents to resist lateral forces and transmit loads to points of support. For certain span lengths, permanent intermediate diaphragms shall be provided to stabilize the beams during construction.

To simplify the bill of materials, all longitudinal and transverse reinforcing bars in concrete diaphragms and transverse edge beams, except the #19 threaded bars, may be epoxy coated.

63-7.02 Intermediate Diaphragms

Intermediate cast-in-place concrete diaphragms shall be provided for an I-beam or bulb-tee beam superstructure as follows:

1. For spans greater than 25 m but less than or equal to 40 m, provide diaphragms at the mid-span.

2. For spans greater than 40 m, provide diaphragms at the span third points.

For a structure with spans less than or equal to 25 m, a note should be placed on the plans that states the following:

> *Suitable restraint shall be provided to prevent the rotation of the beams (particularly the outside beam) from construction loads, such as the weight of the concrete deck, finishing machine, forms, etc.*

A note should also be added to the plans that states the following:

> *Concrete in the intermediate diaphragms shall attain a compressive strength of 21.0 MPa before the deck concrete is poured.*

The diaphragms should not be connected to the deck slab to avoid cracking of the deck over the diaphragm. For a skew of less than or equal to 25 deg, the diaphragms should be placed parallel to the skew. For a skew of greater than 25 deg, the diaphragms should be staggered and placed perpendicular to the beams. See Section 63-16.0 for typical details of intermediate cast-in-place concrete diaphragms.

A spread box beam superstructure having an inside radius of curvature of less than 240 m shall have intermediate diaphragms between the individual boxes. The required spacing will depend upon the radius of curvature and the proportions of the webs and flanges. The diaphragms should normally be placed on the radial lines. Other box beam superstructures do not require intermediate diaphragms.

63-7.03 End Diaphragms

Cast-in-place end diaphragms or edge beams are mandatory, except for an adjacent precast concrete box beam superstructure. Integral end bents function as full-depth diaphragms. End diaphragms serve the purposes as follows
:

1. as a perimeter beam for the deck;
2. support the deck joint device; and
3. transfer lateral loads to the end bent.

For typical details of end diaphragms (transverse edges beams), see Section 61-5.03. For typical details of integral end bents, see Section 67-1.01.

63-7.04 Interior Pier or Bent Diaphragms

Cast-in-place diaphragms are mandatory at all interior piers and bents, except for an adjacent precast concrete box beam superstructure. They serve the purposes as follows:

1. transfer lateral loads to the piers or bents, and

2. for beams made continuous for live load, strengthen the cast-in-place closure placement by providing lateral restraint.

The minimum width of diaphragm for bulb-tee beams shall be 900 mm, for I-beams it shall be 750 mm, and for spread box beams, it shall be 600 mm. The clear distance between beam ends shall be 150 mm unless otherwise approved. This dimension should always be determined parallel to the longitudinal centerline of the beam.

See Section 63-16.0 for typical details of cast-in-place concrete pier and bent diaphragms. The information illustrated in the figures therein is as follows:

1. diaphragm widths;
2. diaphragm reinforcement;
3. cap keyway details;
4. clear distance between adjacent beam ends;
5. bearing pad location details;
6. cap sizing details; and

7. beam threaded bar hole/insert location details.

The figures in Section 63-16.0 also show bearing layouts for skewed structures with I-beams, bulb-tee beams, or box beams. For essentially the same skew angle, the bearing pads are oriented differently in these figures. The ideal orientation of the pads, somewhere between the direction of the beams and the normal drawn to the bearing line, is a function of the skew, the length-to-width ratio, the component rigidities, and the position of loads. In reality, the structural significance of the orientation is small and geometric requirements should govern.

63-8.0 TRANSVERSE CONNECTION OF PRECAST BOX BEAMS

Adjacent, precast, prestressed box beam bridge superstructures have been used extensively. Historically, the shear keys in this type of superstructure tend to crack and leak even with a thin concrete deck placed composite with the beams. Research indicates that these cracks are mainly due to thermal forces and not due to the live load of a vehicle moving across the bridge.

Research has shown the following method is effective to minimize cracking in the shear keys between the beams.

1. Use epoxy grout due to its high bond strength.

2. Use a full-depth shear key to stop the joint from performing like a hinge to prevent the joint from opening. In past designs, the area below the key was open and free to rotate. With this area grouted, the movement of the joint will be reduced.

3. Apply compression across the joint by means of transverse tensioning rods. This will help prevent opening of the joint.

Figures 63-8A, 63-8B, and 63-8C illustrate methods to minimize cracking in this type of structure.

The joints between the beam shear keys, as well as the recesses for the transverse tensioning rods on the exterior face of the beam, should be grouted with an epoxy grout as shown in the details.

After the joints between the beams are grouted, a preliminary tightening of the transverse tensioning rods should be performed. Once this is completed, a final tensioning of the rods should be performed to yield 138 MPa as developed by a torque of 271 N-m.

63-9.0 SEGMENTAL CONSTRUCTION

Bridge designers are continually being challenged to design structures with long spans and low initial cost. Long spans are often used to reduce the number of piers required for a water crossing, and for the elimination of piers adjacent to roadway shoulders at an overpass bridge.

Prestressed concrete beam lengths in the range of 30 m to 40 m have become fairly common. For a continuous structure, the girders are most often fabricated in lengths to span from support to support. A closure pour is then made over the piers to provide continuity for live load and superimposed dead loads. This type of construction is very cost effective because the girders can be erected in one piece without falsework. However, if girders are too long or too heavy to be shipped in lengths to accommodate the spans, spliced girders or segmental construction is an option. Construction techniques have been developed that reduce the cost and, can often make concrete girders competitive with steel girders for spans in excess of 80 m. The most commonly used techniques are as follows:

1. segmental post-tensioned box girders erected on temporary falsework or by the balanced cantilever method; and

2. precast concrete girders spliced at the construction site. These girders can either be supported on temporary falsework or spliced on the ground and lifted into place on the supports.

Most spliced-girder bridges have bulb-tee beams with post-tensioning. This shape has been chosen due to its lightweight and economical cross section.

For staged construction, cambers, deflections, and end rotations of the structural components should be accurately calculated during the various stages of construction.

For further information, see publications of the Precast/Prestressed Concrete Institute, Post-Tensioning Institute, and the Segmental Concrete Bridge Institute. Another source of information is the AASHTO *Guide Specifications for the Design and Construction of Segmental Concrete Bridges*.

63-10.0 SEMI-LIGHTWEIGHT CONCRETE

Normal-weight concrete has a unit weight of between 2240 kg/m^3 and 2400 kg/m^3. The use of semi-lightweight concrete, with normal-weight sand mixed with lightweight coarse aggregate, is permitted with a specified density of between 1920 kg/m^3 and 2080 kg/m^3. Other unit weights may be used if approved by the Design Division Chief. This concrete has mainly been used in long-span, bulb-tee beams where weight reduction is important for shipping or handling, and the extra cost of the semi-lightweight concrete is economically justified.

The structural performance of this concrete is generally equal to that of normal-weight concrete. However, the potential problems that should be addressed in a special provision are the control of the water content in the lightweight aggregate and the frost-sensitivity of lightweight aggregate for a period of two weeks after casting. Consideration must be given to using mix design procedures for lightweight concrete as described in ACI 211.2.

The modulus of elasticity will be less than that for normal-weight concrete. Creep, shrinkage, and deflection must be appropriately evaluated and accounted for if semi-lightweight concrete is to be used. The designer should request and obtain pertinent data from the Materials and Tests Division on the shrinkage, creep, modulus of rupture, permeability, coefficient of thermal expansion, and freeze-thaw resistance for new mix designs that the designer does not have experience with. If the formula shown in LRFD Article 5.4.2.6 is used in lieu of physical test values for modulus of rupture, the formula for sand-low-density concrete shall be used for semi-lightweight concrete.

63-11.0 DIMENSIONING PRECAST BEAMS

If a precast beam is to be placed on a longitudinal slope, its manufactured dimensions should be modified to accommodate the geometric consequences of the gradient. The casting bed is always horizontal. Consequently, the out-to-out beam length becomes L_{CL}, as follows
:

$$L_{CL} = L/\cos\theta$$

where:

θ = arctan(S/100) = angle of slope of the beam
L = length of beam as it appears in plan view
S = slope of beam as shown in elevation view, in percent

For example, a 36-m long beam with a 5% slope will require an additional length ($L_{CL} - L$) of 45 mm. As shown in Figure 63-11A, the plans and the shop drawings should indicate dimensions L, L_{CL}, a, and b. The seat surfaces are always horizontal and the end surfaces are always vertical once the beam is in place.

Maintaining vertical end surfaces of the in-place beams often only has a minimal effect on the constructability of this type of superstructure, and need only be considered where the dimension b in Figure 63-11A exceeds 40 mm.

If the slope of the beam between supports is more than 1.0%, a beveled recess will be required in the bottom of the beam at the supports. For integral end bents, a steel sole plate cast into the beam recess will be required. The recess in the bottom flange of the beam shall have a minimum recess dimension of 6 mm. The minimum concrete cover over the prestressing strands at the opposite end of the recess shall be 25 mm as shown in Figure 63-11A. For a severe grade where use of the minimum 6-mm recess results in less than 25 mm of cover, either the beam seat should be sloped, or the bottom strand clearance should be increased in 6-mm increments until the 25-mm cover is achieved.

For a beam length in excess of 25 m, the length prior to release of the prestressing strands should be increased due to the elastic shortening, creep, and shrinkage anticipated to occur prior to casting the deck slab. Due to variables beyond the designer's control, the beam fabricator is responsible for making this change.

To avoid sharp corners which may be damaged during construction due to a skew of 15 deg or greater, a chamfer of at least 75 mm width should be placed at each acute corner of prestressed box beams.

63-12.0 OTHER DESIGN FEATURES

63-12.01 Skew

The behavior of a skewed bridge is different than that of a square one. The differences are largely proportional to the skew angle. Although normal flexural effects due to live load tend to decrease as the skew angle increases, shear does not, and there is a considerable redistribution of shear forces in the end zone due to the development of involuntary negative moments therein. For a skew angle of less than 30 deg, the skew may be ignored, and the bridge may be analyzed as a square structure whose span lengths are equal to the skewed span lengths.

LRFD Articles 4.6.2.2.2e and 4.6.2.2.3c provide tabulated assistance to roughly estimate these live-load effects. The factors shown in these tables can be applied to either a simple span or a continuous span skewed bridge. The correction factors for shear only apply to support shears at the obtuse corner of exterior beams, and shear in portions of the beam away from the end supports need not be corrected for skew effects.

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

63-12.02 Shortening of Superstructure

For a long continuous structure, the shortening of the superstructure due to creep, shrinkage, temperature, and post-tensioning (if applicable) should be considered in the design of the beam supports and the substructure.

63-13.0 AASHTO I-BEAMS

Figures 63-13A through 63-13L show details and section properties for these beams.

63-14.0 INDIANA BULB-TEE BEAMS

Figures 63-14A through 63-14EE show details and section properties for these beams.

63-15.0 INDIANA COMPOSITE AND NON-COMPOSITE BOX BEAMS

Figures 63-15A through 63-15R show details and section properties for these beams.

63-16.0 MISCELLANEOUS DETAILS

Figures 63-16A through 63-16V show details for diaphragms, closure pours, support cap sizing, and bearing pad layouts for I beams, bulb-tees, and box beams.