Section 9
Prestressed Concrete

9.1 Introduction

Concrete is approximately ten times stronger in compression than in tension. Typical reinforced concrete design assumes that concrete under tensile loads will crack, and steel reinforcing bars are used to carry the tensile forces. Prestressed concrete design, on the other hand, applies compressive force to the anticipated tension zones of the concrete member by using pretensioned or post-tensioned, high-strength steel strands. When properly designed, tension in the member under service loads is reduced or eliminated and concrete cracking is reduced.

Precast concrete members are especially advantageous in situations where quick erection is desired. Precast concrete members are fabricated year-round and can be delivered, erected, and put into service in a very short time. All prestressed concrete beams are produced using high strength, high performance concrete. The corrosion resistance of the prestressed beams is further enhanced by the addition of 5 gal/cubic yd. of 30% calcium nitrite corrosion inhibitor and two coats of silane sealers. These beams are expected to provide long, maintenance-free service.

A number of prestressed concrete bridge types are used in New York. Although adjacent box beams are the most commonly used, designs using I-beams and bulb-tee sections are also becoming common.

9.1.1 Pretensioning

Pretensioning a concrete member is accomplished by tensioning prestressing strands to the required tensile stress using external jacks and anchors, casting the concrete member around the tensioned strands and, releasing the external strand anchors after the concrete has achieved the required minimum strength. Precompression is induced by the transfer of force through the bond between the prestressing strands and concrete.

9.1.2 Post-Tensioning

Post-tensioning a concrete member is accomplished by tensioning unbonded prestressing strands using an external jack on one end of the member and an anchor placed directly against the hardened concrete on the other end. The strands are typically internal to the member, but may be placed externally. A second anchor is secured against the member and the jacking force is released to transfer the load into the member as a precompression force.
9.2 Adjacent Prestressed Units

The typical three types of prestressed members used for adjacent prestressed unit superstructures are solid slab units, voided slab units, and box beams. The design concept for these types of units is identical and the only differences between them are the member depth, shape of the voids (if any), and the casting procedure.

Adjacent prestressed concrete slab units and box beams are especially appropriate at stream crossings having limited freeboard because they provide a continuous flat surface along the bottom of the superstructure that prevents debris from becoming trapped under the bridge and impeding the hydraulic flow. In addition, their relatively shallow depth provides greater clearance than spread beam types of superstructures.

However, because interior deterioration can not be observed by a visual inspection and they are difficult to repair the use of adjacent prestressed concrete slab units and box beams should be limited to situations where they are essential. For example: stream crossings having limited freeboard.

Type F NEXT beams and Deck Bulb Tee girders can also be placed adjacent and should have either a composite slab or asphalt overlay for a riding surface.

9.2.1 Unit Width

Standard box beam and slab units are available in widths of 4 ft. and 3 ft. Designs that use the fewest number of beams for a given superstructure will achieve the greatest economy in fabrication, shipping, and erection costs. Therefore, even if it results in a wider superstructure than is actually required, an adjacent precast concrete unit superstructure should be made exclusively out of 4 ft. wide units. A combination of 4 ft. and 3 ft. wide units may be selected if the required construction staging sequence or other constraint prevents the exclusive use of 4 ft. units. Configurations involving a single 3 ft. unit mixed with 4 ft. units are inefficient to fabricate and should be avoided. The overall beam deck width shall be shown on the contract plans.

4 ft. wide units should be used for the fascia beams to provide adequate space for the placement of the bottom railing anchor plates or concrete barrier reinforcing bars. This is especially important for alignments requiring curved railing or barrier.

Type F NEXT beams are available in widths between 8'-0' and 12'-0".

9.2.2 Unit Depth

Typical prestressed sections are shown on the BD sheets. For multi-span bridges, a constant unit depth is preferable across all of the spans since variable depth units are difficult and expensive to construct. BD sheets for NEXT beams are currently under development. Designers should contact the Concrete Engineering Unit for recommended details until those BD sheets are adopted.
9.2.3 Deck Overhangs

Overhangs on the reinforced deck of adjacent prestressed slab units and box beams shall be a minimum of 4” and a maximum of 6”. Overhangs less than 4” require approval of the D.C.E.S. Overhangs greater than 6” are not allowed. The bottom of the overhang shall slope to drain away from the beam so that chloride-laden runoff water will not run down the side of the beam.

Overhangs for Type F NEXT beams shall be limited to 3'-6”.

9.2.4 Longitudinal Joints

The standard longitudinal joint size between adjacent prestressed slab units and box beams shall be a minimum of ¾” and a maximum of 1½” at the bottom. The use of larger joints requires approval of the D.C.E.S. Joints between stages of stage construction shall follow the details shown on the BD sheets.

The standard longitudinal joint size between adjacent Type F NEXT beams shall be ½” at the top of the beam.

9.2.5 Skew

The designer should make every reasonable effort to reduce or eliminate bridge skew. This may require early discussions with highway design personnel. The maximum allowable skew angle for a bridge using box or slab units is 50°. Larger skews require approval of the D.C.E.S.

The maximum allowable skew angle for a bridge using Type F NEXT beams is 15°. Larger skews require approval of the D.C.E.S.

9.2.6 Diaphragms and Transverse Tendons

Internal diaphragms in adjacent precast concrete slab units and box beams shall be positioned parallel to the skew and have a minimum width of 1'-2”. Transverse tendons shall also be placed parallel to the skew of the unit and be placed as close to the middepth of the section as possible. Each transverse tendon consists of three ½” diameter low relaxation strands tensioned to 28,000 lb. per strand. Transverse tendons are tensioned after the shear keys have been grouted and before the deck slab has been placed.

Internal diaphragms and transverse tendons within precast units shall be spaced as follows:

- For span lengths less than 50 ft., a total of three transverse tendon locations are required. One group of tendons is located at each end of the unit approximately 7” from the centerline of bearings and another group of tendons is located at the centerline of the span.
For spans greater than or equal to 50 ft., a total of five tendon locations are required: one group of tendons at each end approximately 7" from the centerline of bearings, one group of tendons at the centerline of the span, and one group of tendons midway between each end group and the centerline of the span.

For stage construction placing of transverse tendons and diaphragms shall be as shown on the appropriate BD sheets.

The transverse tendon holes in all units and the transverse tendon blockout on the fascia units should be checked to ensure that they do not interfere with either the longitudinal prestressing strands or bar reinforcement.

NEXT beams do not require any diaphragms or transverse tendons.

9.3 Spread Precast Concrete Beam Superstructures

Although a spread precast concrete beam superstructure requires a thicker concrete deck with heavier reinforcement and the necessary form work for the deck placements, the reduced number of beams used per span may prove economical for spans up to 100 ft. In addition, bridge superstructures utilizing spread concrete beams have some advantages over adjacent precast concrete unit superstructures:

- Spread beams have open bays to accommodate utilities when required.
- Spread beams can accommodate field adjustments due to variations in camber and/or camber growth, especially for staged construction.
- Spread beams are better suited to handle large deck cross slopes and curved alignments.

9.3.1 Spread Prestressed Box Beams

The provisions of Section 9.2 of this manual shall apply except as specifically noted below:

- Only 4 ft. wide box units should be used. Alternate widths require approval by the D.C.E.S.
- For spread box units, external diaphragms shall be placed as follows:
  End diaphragms are always required.
  No intermediate diaphragms are required for spans up to 65 ft.
  One intermediate diaphragm is required at midspan in spans over 65 feet and up to 100 feet. The external and internal diaphragms at midspan must be on the same line.
  It is not anticipated that spread boxes will be used over 100 feet. I-girders should be used for spans over 100 feet.
- Contract plans and beam details shall show provision for attaching reinforcement in cast-in-place or precast concrete diaphragms to the spread box beams.
9.3.2 Prestressed I-Girders

The preferred I-Girder shape is the Bulb-Tee.

- The framing plan for prestressed I-girders shall be as shown on the appropriate BD sheets. Contract plans shall normally show only galvanized steel diaphragms. The contractor shall be allowed to substitute cast-in-place diaphragms or precast concrete diaphragms as options. Occasionally, the designer may require concrete diaphragms because of site specific conditions. The cost of diaphragms shall be included in the cost of the beams. All inserts for diaphragm connections adjacent to a deck joint shall be stainless steel.

- No intermediate diaphragms are required for spans up to 65 ft. Midspan diaphragms are required for spans greater than 65 ft., and up to 100 ft. Spans greater than 100 ft. require diaphragms at the third points.

- For superstructures with cross slope greater than 4%, AASHTO I-beams should be considered. These shapes have narrower top flanges, which will eliminate the need for large haunches. Bulb-Tee beams may also be used by reducing the top flange down to three feet in width.

9.3.3 Type D NEXT Beams

Type D NEXT Beams are formed with a full structural deck section composite with the two concrete tee beams and require a closure pour between beams. The actual closure pour width depends upon the closure pour material. The closure pour utilizes Ultra High Performance Concrete (UHPC) or HP concrete between the Type D NEXT Beam sections since there are no transverse tendons.

Type D NEXT beams are available in widths between 8'-0' and 10'-0".

NEXT beams do not require any diaphragms or transverse tendons.

The maximum allowable skew angle for a bridge using Type D NEXT beams is 30°. Larger skews require approval of the D.C.E.S.

Overhangs for Type D NEXT beams shall be limited to 3'-6".

BD sheets for NEXT beams are currently under development. Designers should contact the Concrete Engineering Unit for recommended details until those BD sheets are adopted.
9.4 Segmental Precast Box Girders

Segmental precast box girder superstructures may be viable and economical alternates for the following type of structures:

9.4.1 Long Multi-Span Bridges

Segmental precast box girders are well suited for long multi-span bridges on straight or slightly curved alignments in locations where Temporary Traffic Control issues and/or environmental concerns require that field work be minimized. Repeated use of an erection set up for the box girder segments is the main advantage. The Span-by-Span method of erection is generally used for these bridges.

9.4.2 Long Span Bridge on High Curvatures

Segmental precast box girders are well suited to accommodate high curvatures on long spans due to their high torsional stability. The balanced cantilever method of erection is generally used for these bridges.

9.4.3 Aesthetics

When long open spans with clean visual lines are desired, segmental precast box girder superstructures are a good solution. Haunching of the segmental girders to improve the visual impact and structural efficiency is possible with this type of superstructure.

9.4.4 Durability

The expected durability of segmental box girder bridges is relatively high. Segmental precast box girder bridges utilize post-tensioning in both the longitudinal and transverse directions to be free of tensile cracks. This results in an expected substantial increase in the durability of the overall structure. However, there are areas of vulnerability unique to this type of bridge.

1. Since the deck slab is an integral part of the box girder system, the complete replacement of the bridge deck is nearly impossible. To reduce this risk, the structure should be designed so there is no tensile stress at the top surface of the segment under service load conditions both including and excluding time dependent effects.

2. Deck run-off should not be allowed to flow over the grouted block-outs for tendon anchorages. When end anchorages are located in vulnerable areas, such as beneath a deck expansion joint, additional protective measures shall be provided. Post-tensioning ducts within the deck shall be polyethylene. Fabrication and erection of these structures shall be as per the Prestressed Concrete Construction Manual (PCCM).
9.5 Bearings for Prestressed Concrete Structures

All new prestressed concrete superstructure designs, with the exception of those using integral abutments, require elastomeric bearings of sufficient thickness to ensure that the bottom of the prestressed unit will be above the bridge seat a minimum of $\frac{3}{8}$" for box beams and $\frac{1}{2}$" for slab units. Cement mortar pads shall not be placed under the bearings.

For rehabilitation projects that require mortar pad replacement, the designer should choose one of the following alternatives:

- Replace the existing elastomeric bearings and mortar pads with thicker elastomeric bearings.
- Replace the mortar pad with a galvanized steel plate of equivalent thickness.
- Step the bridge seat or pedestal to an elevation sufficient to provide the necessary clearance (This option will normally require the use of Class DP Concrete, as specified in Section 582 of the NYSDOT Standard Specifications for Construction and Materials).

When choosing an appropriate alternative, the designer should strive for the most cost effective solution.

Bearings must be placed perpendicular to the centerline of the unit. The bearing width, at a minimum, must be $\frac{1}{2}$ the width of the unit measured perpendicular to the centerline of the unit.

When the height difference across the width of the bearing due to camber and grade is in excess of the limitations set in the design specifications, then a tapered bearing (for adjacent box or slab units) or a constant thickness bearing with a tapered sole plate (for Bulb Tees and AASHTO I-beams) matching the required slope must be used.

9.6 Concrete Strength

High-Performance Concrete shall be the standard concrete for prestressed bridge elements. The minimum concrete strength $f'_{cn}$ for prestressed concrete bridge beams shall be 10 ksi. The concrete strength at transfer $f'_{c|t}$ can be taken as $0.7f'_{c}$ unless the designer determines a higher transfer strength is necessary.

9.7 Prestressing Strand Type

Only 270ksi Low-Relaxation Prestressing Steel Strand shall be used. The standard diameter used by NYSDOT is 0.6 inch. Other diameters are available, but may only be used with approval of the D.C.E.S. Strength requirements and areas for the strand are available in ASTM A416.
9.8  Strand Pattern for Pretensioned Elements

9.8.1  Precast Box and Slab Units

A 2" x 2" center to center grid pattern shall be used for the prestressing strands in prestressed concrete beams. Strands shall not be placed within 2" of the centerline of the beam to provide room for the anchor dowel holes at the end of the beam. Strands shall not be placed such that they will conflict with the transverse tendons or tendon recesses. For additional information, see the appropriate BD sheet.

Prestressing strands shall be distributed evenly across a row to achieve uniform pretensioning in the end zones. Clustering of strands in the bottom corners of beams should be avoided as the uneven stresses can cause distortions to the beam. This is especially critical in longer beams with large skews.

9.8.2  Precast I-Girders, Bulb Tees and NEXT Beams

Prestressing strands are arranged in a 2" x 2" grid pattern as shown on the appropriate BD sheet. Prestressing strands shall be distributed evenly across a row to achieve uniform pretensioning in the end zones.

9.9  Tensile Stresses Due to Pretensioning

If higher than allowable tensile stresses are encountered during the design of prestressed members (usually at the top surface of the beam ends) the following design modifications are suggested in the order of preference:

1. Rearrange the strand pattern, including addition of strands near the surface exhibiting excessive tension. In general, four fully tensioned strands is a reasonable maximum number of strands to be placed near the tension surface for slab units. For box units, six is a reasonable maximum. For Bulb Tees and AASHTO I-Beams, 20% of the total number of strands (not including draped strands) is a reasonable maximum. In all cases, engineering judgment is required.

2. Drape strands for I-Girders (Bulb Tees and AASHTO I-beams).
   
   Note: Where draped strands are used, the total hold down force of all draped strands shall not exceed 75% of the total beam weight.
   
   Note: Prestressing strands in slab units or box units shall not be draped.

3. Debond some prestressing strands at the end of the unit to avoid excessive end stresses. Typically, this is accomplished in the fabrication plant by wrapping strand with a plastic sheath to prevent the bond from developing between the concrete and the prestressing strand.
When debonding of prestressing strands is required, design shall be in accordance with the *NYSDOT LRFD Bridge Design Specifications* with the following criteria:

a. The maximum allowable number of debonded prestressing strands is 25% of the total number of strands.
b. No more than 40% of the number of prestressing strands in any one row may be debonded.
c. The debonding pattern shall be symmetrical about the beam centerline.
d. The spacing of debonded strands shall be a minimum of 4”.
e. The outermost prestressing strands in a row shall not be debonded.
f. The debonded length(s) shall be clearly detailed on the contract plans. A maximum of four prestressing strands are permitted to be debonded for a given length. A minimum difference of 2’-0” is required between debonding lengths.
g. Do not debond prestressing strands in units 1’-3” or less in depth.

4. Provide a reasonable amount of bonded reinforcement as per the provisions of the design specifications.

**9.10 Prestress Losses**

Loss of prestress is the difference between the initial tensile stress in prestressing tendons at the time the strands were seated in their anchorages, and the effective prestress at a particular time at the considered location.

Losses that apply to both pretensioned and post-tensioned elements are Concrete Shrinkage, Elastic Shortening, Concrete Creep, and Steel Relaxation. Losses that apply only to post-tensioned elements are Anchorage Set and Friction (for drape and wobble). Computation of the losses shall be as per the applicable provisions of the design specifications.

**Concrete Shrinkage** - Shrinkage, after hardening of concrete, is the decrease with time of concrete volume. The decrease is due to changes in the moisture content of the concrete and physical-chemical changes, which occur without stresses attributable to actions external to the concrete. Shrinkage is conveniently expressed as a dimensionless strain under steady conditions of relative humidity and temperature.

**Elastic Shortening** - The concrete beam shortens at transfer when the prestressed strands are released and the force in them is transferred to the concrete. This elastic shortening is immediate and results in a reduction in the strain of the prestressing steel and therefore a prestress loss. The loss from elastic shortening should be included in both initial and total loss computations.

**Concrete Creep** - The time dependent increase of strain in hardened concrete subjected to sustained stress is defined as concrete creep.

**Steel Relaxation** - Steel relaxation is very similar to concrete creep. With steel relaxation the length of the strand is held constant under stress and there is a time dependent loss in stress.
The designer shall use a “$t$” of 18 hours for computing steel relaxation loss at transfer. This represents the shortest time that is likely to occur between jacking and detensioning. For initial stresses the main problem is overstressing the beam ends due to excessive prestressing force.

**Anchorage Set** - Some loss of prestress occurs to post-tensioned tendons as the anchorage hardware deforms and sets at the transfer of tension. The amount of set is a function of the type of anchorage system used. The amount of prestress loss is a function of this anchorage set and the length between anchorages. Power seating of the chucks tends to reduce this loss. For design purposes assume anchor set as $\frac{3}{8}$ in.

**Friction** - Tendons also lose some prestress due to friction inside the ducts during stressing operations.

**Total Losses** - Some of the losses mentioned above are interdependent. Shrinkage and concrete creep reduce the strain in the prestressing steel, which reduces the force in the prestressing steel. The reduction in force in the prestressing steel affects elastic shortening, future concrete creep and steel relaxation.

### 9.11 Allowable Stresses

#### 9.11.1 Temporary Stresses

Temporary stresses correspond to the stresses that are present at transfer. Only initial losses should be considered when checking these stresses.

#### 9.11.2 Final Stresses

Final stresses represent the stresses at service load after all losses have occurred. When AASHTO HL-93 live loading is used, the maximum allowable tension in the precompressed tensile zone shall be $0.0948 \sqrt{F'_c}$ ksi, as per the LRFD specifications. When the NYSDOT Design Permit Vehicle is used the maximum allowable tension shall be $0.14 \sqrt{F'_c}$ ksi. The design allowable stresses shall be shown on the contract plans.

### 9.12 Reinforcement

Reinforcement in prestressed units shall not be epoxy coated with the exception of the composite stirrups extending into the deck, or the top longitudinal bar extending into the approach slab which may be epoxy coated or galvanized.

#### 9.12.1 Shear Stirrups

Detailing of shear reinforcement shall follow the guidance shown on the appropriate BD sheets.
9.12.2 Composite Design Reinforcement

Composite flexural members consist of prestressed members acting with a cast-in-place concrete deck. In order for the deck to act compositely, reinforcement must be provided extending out of the beams into the slab to resist the horizontal shear developed across this plane. Composite shear reinforcement shall be provided for the full length of prestressed concrete bridge beams, including the negative moment areas of continuous spans.

9.12.3 Anchorage Zone Reinforcement

When pretensioned strands are released and their stress is transferred to the hardened concrete bonded to the strands, the concrete at the beam ends experiences tensile stress perpendicular to the direction of prestressing. Anchorage zone reinforcement shall be provided to resist these stresses. For slab units and box beams, stirrups with multiple legs can be used to accommodate required reinforcing within the specified distance from the end of the beam.

9.13 Camber

Due to the eccentric nature of prestressing, prestressed concrete units are typically curved upward under low values of externally applied loads. The resulting upward deflection is called camber. Camber may increase or decrease with time, depending on the stress distribution across the member under sustained loads.

Units shall be designed so that the algebraic sum of the beam camber at prestress transfer due to prestress force, the beam dead load deflections due to non-composite dead load, and superimposed dead load deflections due to applied superimposed dead loads results in a positive (upward) camber. The dead load from a future wearing surface shall be included in the determination of camber.

Allowed camber deviations for beams (see Section 7 of the PCCM.) should be considered in determining minimum expected camber based on design calculations. The following minimum net positive cambers are recommended:

- Spans 80 ft and above: ½” minimum
- Spans 50 ft to 80 ft: ¼” minimum
- Spans less than 50 ft: ⅛” minimum

The contract plans shall show the camber at prestress transfer and the deflections due to non-composite dead load and superimposed dead load.

9.14 Stage Construction Camber Differences

For a given project, fabricators typically cast all of the beams of a given size at the same time to minimize the time required to set up the casting beds. If these beams are subsequently erected at the same time, differential camber between beams is rarely a significant problem.
On stage construction projects, the precast beams may be fabricated at relatively the same time and erected many months, even years apart. The haunch provided for spread prestressed box beams, AASHTO I-beams and Bulb Tees is sufficient to accommodate this differential camber growth and need not be considered. Since adjacent precast unit superstructures have no haunch, the differential camber due to time dependent effects shall be considered.

The anticipated camber growth during storage of Stage 2 units may be assumed to be 50% of the camber at transfer. For all staged construction bridge superstructure projects, the minimum Stage 1 deck slab thickness shall be 7” in order to provide a minimum 6” deck slab over the Stage 2 units. The additional Stage 1 slab thickness of 1” shall be considered as extra dead load in the unit design calculations.

If the anticipated camber growth with no control measured during storage is greater than 1”, specific measures to control camber growth of the Stage 2 beams or other methods to limit the different camber growth between Stage 1 and Stage 2 must be specified in the contract documents. Typical notes in Section 17 of this manual must be placed on the contract plans.

Example:
Camber at transfer (w/o creep) = 1 inch
Anticipated camber growth = 0.5 x 1 inch = ½ inch

9.15 Continuity Design at Interior Supports

Unless significant differential settlement between supports is expected, all multi-span prestressed concrete superstructures shall be designed and detailed with continuity connections at all interior supports except at locations on long structures were periodic joints are needed to accommodate thermal movements.

Other than post-tensioned structures, multi-span prestressed concrete superstructures shall be designed and detailed as continuous for live load only. All multi-span prestressed concrete superstructures with post-tensioned and spliced beams shall be designed and detailed as continuous for all live and dead loads. See Section 9.17 for guidance on post tensioned structures.

Current practice for continuous for live load design is to establish the continuity connection at the same time as the placement of deck concrete. Hence, dead load due to the deck concrete will be handled by prestressed beams acting as simply supported beams. Live loads and superimposed dead loads applied after the deck concrete hardens will be handled by the continuous composite (beams and deck acting together) structure. Loads applied after the hardening of the deck concrete will cause negative moment over the continuity connections.
The use of high-strength, high-performance concrete (10 ksi) for all prestressed concrete bridge beams and requiring the beams age for a minimum age of 60 days prior to deck placement helps reduce stresses on the continuity connection. However it is not possible to accurately predict all of the loads applied on the continuity connections. The unknown number of actual field loading conditions and the number of assumptions needed makes it difficult to design the continuity connection in full accordance with the requirements of the LRFD Design Specification. Therefore, designers of prestressed concrete beams designed and detailed with continuity connections shall adhere to the following guidance:

- The beams shall initially be designed as simple spans, neglecting the effect of continuity, including full live load and superimposed dead load. The design shall then be analyzed assuming full continuity for the live load plus impact loads, and for the superimposed dead load. The final beam design shall satisfy the requirements for both conditions.
- Designers shall assume full continuity in calculating the tensile stresses in the deck and at the top of the beam ends. The designer shall ensure that there is adequate top mat reinforcement in the slab to resist this condition.
- All Load Ratings shall be calculated on the simple span condition only.
- NYSDOT standard continuity connection details may be used without design of the continuity connection.

Continuous for live load details for box beams and slab units can be found on BD-PA10E. Continuous for live load details for bulb tee and AASHTO I-beams can be found on BD-PS8E.

9.16 Corrosion Inhibitors and Sealers

Prestressed concrete elements shall use corrosion inhibitor and penetrating silane sealer. See the PCCM for details.

9.17 Post-Tensioned Spliced Girder Designs

Prestressed concrete bridge beams may be spliced by joining two or more beam segments to form one beam. Typically, splicing is achieved by cast-in-place concrete along with longitudinal post-tensioning. Splicing of bridge beams is generally used for one or more of the following reasons:

- Increasing span lengths to reduce the number of sub-structure units and total project cost;
- Increasing the girder spacing to reduce the number of girder lines and total project cost;
- Increasing span lengths to improve safety by eliminating shoulder piers or interior supports;
- Minimizing structure depth through the use of long, continuous members to obtain required vertical clearance for traffic, waterways, and so forth;
- Avoiding the placement of piers in water to reduce environmental impact and total project cost;
- Placing piers to avoid obstacles on the ground, such as railroad tracks, roadways, and utilities;
• Improving aesthetics through various design enhancements, such as more slender superstructures, longer spans, of haunched sections at piers;
• Eliminating joints for improved structural performance, reduced long-term maintenance/increased service life, and improved rideability.

Whenever possible, part of the longitudinal post-tensioning shall be applied after the hardening of the deck concrete so that net tension on top of the deck surface is less than or equal to the modulus of rupture.

The Contract plans shall show a recommended installation method and post-tensioning sequence. See current BD Sheets for additional guidance. The structural analysis should consider the effects of fabrication and erection tolerances on bridge performance.