8-15-2003

# Bridge Design Guide, Chapter 6 : Concrete, 2003

Maine Department of Transportation

# Chapter 6

# CONCRETE

**Hancock-Sullivan Bridge, Hancock-Sullivan**

**Ogunquit Beach Bridge, Ogunquit**

# 6 CONCRETE

## 6.1 Precast Concrete

### *6.1.1 General*

#### 6.1.1.1 Prestressing

To control stresses at the ends of prestressed beams, harping or debonding of strands may be specified. Harped strands allow for a more favorable distribution of stresses in the beam resulting in more efficient use of strands. Debonding does not allow the same degree of flexibility to control stresses throughout the beam. Every effort should be made to keep a straight strand pattern with a few debonded strands. However, in some cases, particularly with deep members such as New England Bulb Tees (NEBT), harping may be necessary. If harped strands are specified, the Structural Designer should be familiar with the practice and limitations of regional producers. For further guidance on this subject, consult with the Bridge Quality Assurance Team.

#### 6.1.1.2 Camber, Deflection, and Blocking

The Structural Designer should consider camber, deflection, and blocking to avoid negative blocking in the structure. In the past, minimum practical haunch dimensions above the centerline of the top of the beam were found to be 2 inches for AASHTO I-girders and 3 inches for NEBT girders or spread box beams. These dimensions should be checked to determine their applicability to the design and increased if needed to avoid encroachment into the deck by the top flange.

Camber of the beams must be considered with the proposed centerline profile. Differences between the camber and the roadway profile can lead to a haunch or leveling slab that is too thin or one that is excessively thick. Methods for limiting or increasing camber can be found in PCI Precast Prestressed Bridge Design Manual (1997).

For staged construction projects and projects where beams may be fabricated more than four months prior to placement, the Structural Designer must consider the effects of camber growth.

Dead load deflections and a table of bottom of slab elevations should be given on the contract drawings for spread box, NEBT, and AASHTO I-girder structures. The Contractor should use screed rails to construct the finish grade, as noted in Appendix D Standard Notes Precast Concrete Superstructures.

#### 6.1.1.3 Section Properties

Section properties should be based on the concrete section alone, neglecting any effect of mild reinforcing steel or prestressing strand.

When designing precast beams composite with a concrete deck, section properties should be computed assuming a haunch dimension of one inch and an equivalent transformed width of deck. Dead loads should be based on the actual concrete dimensions.

#### 6.1.1.4 Constructibility Check

Special consideration should be given to construction loading and the deck casting sequence. This sequence, including placing of the diaphragms, can directly affect the capacity of the superstructure components. The deck casting sequence should be specified on the plans, and should include instructions for placing diaphragms.

#### 6.1.1.5 Allowable Stresses at Service Loads

When precast prestressed superstructures are used over salt water or other corrosive environments, under no circumstances should the tension at load combination Service III exceed the requirements of AASHTO LRFD Section 5.9.4.2.2, for the severe exposure condition.

### *6.1.2 Materials*

#### 6.1.2.1 Concrete

In general, precast concrete designs should be based on a 28 day compressive strength up to 6.5 ksi. Concrete strengths in excess of this should be used only when approved by the Engineer of Design, and when verified that regional precasters are capable of producing quality concrete at higher strengths. Precast concrete should be specified as Class P on the plans. The maximum permeability for the precast concrete should be indicated on the plans, which is 3000 coulombs in most cases.

Prestressed concrete units should contain a calcium nitrate corrosion inhibitor admixture, commonly referred to as DCI, in the concrete mix at a rate of 3 gal/yd$^3$. This requirement is specified in Standard Specification Section 535 - Precast Prestressed Concrete Superstructure. For structures over salt water or other corrosive environments, the Structural Designer should increase the rate of corrosion inhibitor to 5.5 gal/yd$^3$.  The Structural Designer must verify that the PS&E package contains a Special Provision for this requirement.

#### 6.1.2.2 Prestressing Strand

Prestressing strand should be uncoated low relaxation seven wire strand meeting the requirements of AASHTO M 203 Grade 270.  Strands for NEBT structures should typically be 1/2" diameter, with a maximum 0.6" diameter. Strands for precast deck panels should be a maximum 3/8" diameter, while all other strand should be a maximum diameter of 1/2".

Prestressing bars should be uncoated high strength steel bar meeting the requirements of AASHTO M 275.

#### 6.1.2.3 Mild Reinforcement

Refer to Section 6.2.1.2 Reinforcing Steel for reinforcement material requirements for non-prestressed reinforcement.

### *6.1.3 Economy*

#### 6.1.3.1 Release Strength

Concrete strength at release of prestress force can significantly affect cost. Precasters rely on daily use of their prestressing beds.  Concrete strength at release is often the controlling factor in the concrete mix design. Excessive release strengths will either force the precaster to use higher strength concrete than the design requires or delay the release of prestressing force.  The suggested release strength should be in the range of 4 to 4.5 ksi.

#### 6.1.3.2 Beam Sections

When designing precast superstructures, uniform beam widths and strand patterns should be used whenever possible.  Prestressing beds are long and can often accommodate more than one beam.  Uniform beam widths and strand patterns allow more than one beam to be placed in the prestressing bed at a time, thus accelerating and economizing production.

### *6.1.4 Design Requirements*

#### 6.1.4.1 Concrete Cover

All precast main carrying members should be designed with the stirrups encasing all prestressing strands. The minimum cover for the stirrup is 1 inch from the bottom of the section.

#### 6.1.4.2 Voided Slab and Butted Box Beam Bridges

A. Transverse Post-Tensioning

Normally, post-tensioning should be accomplished by the use of 0.6” diameter prestressing strand as specified in the applicable Supplemental Specifications. In cases where the chuck-to-chuck length is 25 feet or less, prestressing strand cannot be used due to excessive overstressing for the setting losses.

> *Commentary: The use of 0.6” diameter prestressing strand with a larger post-tensioning force is intended to limit cracking of the shear keys. Standard Detail 535(02) has been reviewed and approved for use with this larger strand size.*

Diaphragms and strand locations should be spaced as described in Table 6-1. Diaphragms and post-tensioning ducts may be placed parallel to the centerline of bearing for skews less than 30°. For skews over 30°, diaphragms should be placed normal to the beams and consideration should be given to torsional loads from sidewalks, future widening, and maintenance of traffic. The end post-tensioning should be located such that it does not interfere with the wingwalls, including allowances made for the post-tensioning jack.

**Table 6-1 Post Tension and Diaphragm Locations**

| **Beam Type** | **Span** | **Ends** | **1/3 points** | **1/4 points and mid-span** | **Single mid-depth strand** | **Top and bottom strand** |
|---|---|---|---|---|---|---|
| Voided Slabs | All | X | X | | X | |
| Box Beams less than 3 ft deep | ≤ 50 ft | X | X | | | X |
| | > 50 ft | X | | X | | X |
| Box Beams 3 ft and deeper | All | X | | X | | X |

B. Wearing Surfaces

Refer to Section 4.7 Membranes for membrane requirements under pavement. Concrete wearing surfaces should be avoided unless a minimum 6 inch composite leveling slab is used.

C. Leveling slabs

In general, an unreinforced leveling slab should be used on all voided slab and butted box beam structures, if not designed with a reinforced composite slab. The minimum thickness is 2 inches at the curb line, and the cross slope matches the finish slope. The minimum thickness of a reinforced leveling slab is 4 inches. In some cases, the leveling slab may be omitted based upon project specific considerations, if approved by the Engineer of Design.

D. Continuity Design

Prestressed girders should be made continuous for the maximum practical length to avoid expansion joints. In general, the design should follow AASHTO LRFD Section 5.14.1.2 - Precast Beams. The Structural Designer is also referred to Oesterle (1989).

1. Negative Moment Over Piers

As a minimum, sufficient continuity steel should be provided to control cracking at the pier in the wearing surface at service loads. Crack control should be checked in AASHTO LRFD Section 5.7.3.4. The following values should be used for the crack width parameter Z:

| | |
|---|---|
| Bituminous with high performance membrane | 170 k/in |
| Concrete wearing surface* | 77 k/in |

*A crack width parameter up to Z = 130 k/in may be allowed with the use of galvanized or epoxy coated reinforcing steel and low permeability concrete.

Crack width parameters of 170 and 77 k/in correspond to approximate crack widths of 0.016” and 0.007” respectively. More refined methods of determining crack width such as the Gergely-Lutz equation for crack width are allowed.

2. Positive Moment Over Piers

As a minimum, sufficient continuity steel should create a reinforced section that resists 1.2 times the cracking moment.

E. Skew

Voided slab and butted box beam superstructures should not be used for bridges with skew angles greater than 45°. Bridges with heavy skews present problems with beam alignment during erection. Heavy skews also increase shear forces at the obtuse corners that may lead to shear key failure. Utilizing these beams with skews greater than 45° requires the approval of the Engineer of Design.

#### 6.1.4.3 NEBT, AASHTO I-Girder, and Spread Box Beam Bridges

A. Diaphragms

Unless supported by integral abutments, end diaphragms should be designed to allow for jacking during future maintenance operations.

B. Continuity Design

*Post-Tensioned Spliced NEBT Girder:* The Structural Designer is referred to the PCI guidelines for post-tensioning and splicing NEBTs.

*Conventionally Reinforced:* The design should follow AASHTO LRFD. The Structural Designer is also referred to PCI (1997) as well as Oesterle (1989). Refer to Section 6.1.4.2D Continuity Design for further guidance.

C. Deck Overhang Limits

To control flexural stresses in the top flange of exterior beams, the overhanging portion of the CIP slab as measured from the edge of the top flange should be limited to 2 feet.

## 6.2 Cast-In-Place Concrete

### *6.2.1 Materials*

#### 6.2.1.1 Concrete

A. Concrete Class

There are four classes of concrete used for cast-in-place (CIP) structures: Class A, Class LP, Class S, and Class Fill. Guidelines on when to use each class are described in Table 6-2. Refer to Standard Specification Section 502 – Structural Concrete for further guidance.

Concrete strengths in excess of those described in the Standard Specifications should be used only when approved by the Engineer of Design, and when verified that regional suppliers are capable of producing quality concrete at higher strengths.

**Table 6-2 Concrete Classes**

| Concrete Class | Concrete Use |
|---|---|
| Fill | Fill |
| LP | Structural Wearing Surfaces, Sidewalks, Curbs, Barriers, End Posts |
| S | Seals |
| A | All Others |

B. Quality Control and Quality Assurance Guidelines

There are three possible methods for specifying the structural concrete for acceptable quality control (QC) and quality assurance (QA): Method A (Statistical Acceptance), Method B (Small Quantity Product Verification), or Method C (conforming to the requirements). Guidelines on the requirements of Method A, Method B, or Method C are described in Table 6-3. Incentives and disincentives are determined from the QA test results. Under Method A and B, if the test results indicate that the concrete quality is less than acceptable limits, the concrete may be removed and replaced at MaineDOT's discretion.

**Table 6-3 Concrete Testing Requirements**

| Method | QA Responsibility | QC Responsibility | Incentive | Disincentive |
|---|---|---|---|---|
| A | MaineDOT | Contractor | X | X |
| B | MaineDOT | Contractor | | X |
| C | Contractor | | | |

A Special Provision 502 must be included in all contracts that will designate under which method each concrete item will be classified (A, B, or C). The Structural Designer, together with the Construction Resident, must decide during the design phase of the project whether to specify Method A, Method B, or Method C concrete. If there is any doubt, guidance from one of the Construction Engineers may be requested.

Guidelines on when to specify Method A, Method B, or Method C are as follows:

Method A should be specified where quality above the specification requirements is of value. Examples of where Method A is appropriate include, but are not limited to: superstructures, decks, sidewalks, curbing, wearing surfaces, barrier, and mast arm traffic signal supports. Method A should also be used on a substructure unit when it is exposed to a sheltered or corrosive environment (such as exposed portions of an overpass), or where surface architectural treatment is used. P, the unit value for pay adjustment purposes, must be provided in the Special Provision that is included in each contract. P values reflect the price per cubic yard for all pay adjustment purposes. P values will be established on an annual basis and should not be based strictly on bid history information.

Method B should be specified where concrete must meet specifications but where there is no value added by quality exceeding the requirements of the specifications. Examples of where Method B is appropriate include, but are not limited to: abutments, approach slabs, retaining walls, piers, footings, seals, box culverts, concrete fill, pipe pile concrete, traffic signal bases, and sign bases when not cantilevered. Selected substructure units should be method A, as discussed above.

Method C concrete should be specified where concrete quality still has to meet the specifications, but the benefits and costs to the Contractor and to the Department to develop and administer a Quality Control Plan, as required by specifications, are not justified. Examples of where Method C concrete is appropriate include: armored joint repairs; surface repairs to wing walls, bridge decks, abutments, piers, and box culverts; and modifications to existing end-posts. This method should not be specified for structural elements that are expected to have a long design life.

#### 6.2.1.2 Reinforcing Steel

Reinforcing steel, both plain and epoxy-coated, should be deformed bars meeting the requirements of AASHTO M 31. In general, the minimum bar size should be #5 for main reinforcing members and #4 for stirrups.

The use of epoxy-coated reinforcing steel is felt to be a cost effective solution to rebar corrosion for selected locations. The following locations in concrete bridge elements should incorporate the use of epoxy-coated reinforcing steel:

A. Substructure

- All pier columns, shafts, and caps of grade separation structures that are within 30 feet of the traveled way, including footing dowels if they extend above the finished grade line

- All abutment bridge seats and front faces of breastwalls of grade separation structures that are within 20 feet of the traveled way, including footing dowels if they extend above the finished grade line

- The front face of all retaining walls and wingwalls of grade separation structures that are within 20 feet of the traveled way, including footing dowels if they extend above the finished grade line

- All substructure units in their entirety, when the bridge passes over salt water

B. Superstructure

- All deck slabs when the bridge passes over salt water

- All deck slabs of continuous steel structures with concrete wearing surfaces

Other locations, as approved by the Engineer of Design, may also incorporate epoxy-coated reinforcing steel where it is considered to be cost effective. In addition, the Engineer of Design may approve elimination of epoxy-coating at locations where it may not be cost effective, due to low traffic volumes and/or low susceptibility to salt intrusion.

### *6.2.2 Decks*

The deck slab should be carried over the abutment backwall under the following circumstances:

- On abutments with fixed bearings when beam depth is less than approximately 4 feet. On roads with low traffic volume, the Structural Designer may choose to carry the slab over the backwall for beams deeper than 4 feet.

- On abutments with expansion bearings for bridges within the following limits:

    1. Spans up to 40 feet with skew up to 45°

2. Spans up to 50 feet with skew up to 40°

3. Spans up to 60 feet with skew up to 35°

4. Spans up to 70 feet with skew up to 30°

5. Spans up to 80 feet with skew up to 20°

Concrete curbs should be placed continuously with extra longitudinal steel in the top of the curb over piers. This additional reinforcement should extend into the positive moment region not less than the development length of the bar. Sidewalks on bridges are treated in the same manner.

When a deck slab on new girders will be built in successive stages, or when staged construction is used to replace an existing slab, a zipper strip should be considered where sufficient width is available to maintain traffic. A zipper strip is a longitudinal concrete closure pour between two successive deck construction stages. A zipper strip is intended to reduce the effect of adjacent live loads during curing and to minimize cracking between stages.

Based upon the anticipated use of completed parts of a structural slab during construction, the Structural Designer may wish to specify that the formwork be designed to carry all or part of the design live load.

When designing the superstructure slab for a multiple span continuous structure with more than 250 $yd^3$ of deck concrete, an optional deck construction joint must be provided for use if the Contractor elects to place the deck concrete in successive placements. Refer to Appendix D Standard Notes Superstructures.

#### 6.2.2.1 Standard Design of Concrete Slab on Steel Girders

Table 6-4 shows the maximum span for a given slab thickness on steel girders and Table 6-5 shows standard reinforcing steel design for concrete slabs in superstructure slab designs. For these tables, a 3 inch bituminous wearing surface with 1/4” membrane was used in the slab designs. The slab design should not be modified for a lighter weight wearing surface. Refer to Figure 6-1 for an explanation of reinforcing details.

This design uses straight bars top and bottom, without the use of crank bars. If precast deck panels will not be offered as an option to the Contractor (refer to Section 6.2.2.3), the Structural Designer may choose to specify crank bars instead.

Extra distribution bars in negative moment areas should be designed in accordance with AASHTO LRFD Section 6.10.3.7 - Minimum Negative Flexure Slab Reinforcement. In general, the requirement will be met if the bottom mat of distribution steel is as shown in the standard design in Figure 6-1 and the top mat of distribution steel is changed to accommodate the increased requirement for steel. If possible, the top mat of steel should have only one size bar with a minimum spacing of 6 inches.

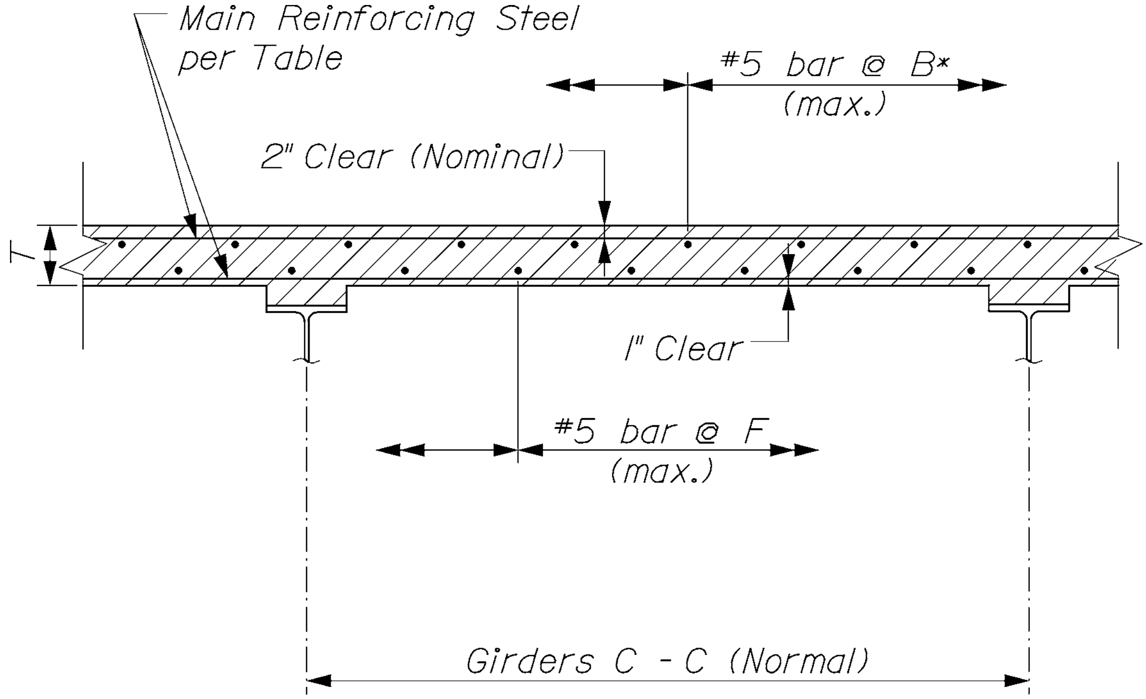
**Table 6-4 Maximum Deck Spans on Steel Girders**

| Slab Thickness (in) | Maximum Girder Spacing | | |
|---|---|---|---|
| | Skew (θ) | | |
| | 0°<θ ≤10° | 10°<θ ≤25° | θ = 0°, θ>25° |
| 7 | 7’-1” | 8’-4” | 7’-3” |
| 7 ½ | 8’-7” | 8’-10” | 8’-9” |
| 8 | 9’-4” | 9’-6” | 9’-6” |
| 8 ½ | 9’-10” | 9’-11” | 10’-0” |
| 9 | 10’-4” | 10’-5” | 10’-6” |
| 9 ½ | 10’-10” | 11’-1” | 11’-0” |
| 10 | 11’-1” | 11’-6” | 11’-3” |
| 10 ½ | 11’-7” | 12’-2” | 11’-9” |
| 11 | 12’-0” | 12’-10” | 12’-3” |

**Table 6-5 Standard Slab Designs**

| Slab T (in) | Main Reinforcing | | | Dim F (in) | Dim B (in) |
|---|---|---|---|---|---|
| | Skew (θ) | | | | |
| | 0°<θ ≤ 10° | 10° < θ ≤ 25° | θ = 0°, θ>25° | | |
| 7 | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 18 |
| 7 ½ | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 18 |
| 8 | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 18 |
| 8 ½ | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 18 |
| 9 | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 18 |
| 9 ½ | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 15 |
| 10 | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 7 | 15 |
| 10 ½ | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 8 | 15 |
| 11 | #5 @ 6” | #5 and #6 alternating @ 6” | #5 @ 6” | 8 | 15 |

Note: The spacing for the main reinforcing steel is measured along the centerline of beam. The main reinforcing steel is parallel to the skew for skews less than or equal to 25° and perpendicular to the girders for skews greater than 25°.




TRANSVERSE SECTION

* For extra distribution steel in negative moment areas, see text.

DESIGN: Load & Resistance Factor Design per AASHTO LRFD - Specifications for Highway Bridges 1998 and Interim Specifications through 2002

LOADING: HL-93 Modified for Strength I (Impact = 33%)

STRESSES: Reinforcing steel ~ fy = 60,000 psi
Concrete ~ f'c = 4,350 psi (Cast - in - place)

**Figure 6-1 Concrete Slab on Steel Stringers**

6.2.2.2 Standard Design of Concrete Slab on Concrete Girders

Table 6-6 shows the maximum span for a given slab thickness on concrete girders and Table 6-5 shows standard reinforcing steel design for concrete slabs in superstructure slab designs. This table assumes a 3 inch bituminous wearing surface with 1/4” membrane, and a minimum top flange width of 3 feet. The slab design should not be modified for a lighter weight wearing surface. Refer to Figure 6-1 for an explanation of reinforcing details.

**Table 6-6 Maximum Deck Spans on Concrete Girders**

| Slab Thickness (in) | Maximum Girder Spacing | | |
|---|---|---|---|
| | Skew (θ) | | |
| | 0°<θ ≤10° | 10°<θ ≤25° | θ = 0°, θ>25° |
| 7 | 8'-10" | 9'-9" | 9'-0" |
| 7 ½ | 9'-10" | 10'-2" | 10'-0" |
| 8 | 10-7" | 10'-8" | 10'-9" |
| 8 ½ | 11'-1" | 11'-4" | 11'-3" |
| 9 | 11'-6" | 12'-0" | 11'-9" |
| 9 ½ | 12-0" | 12'-5" | 12'-3" |

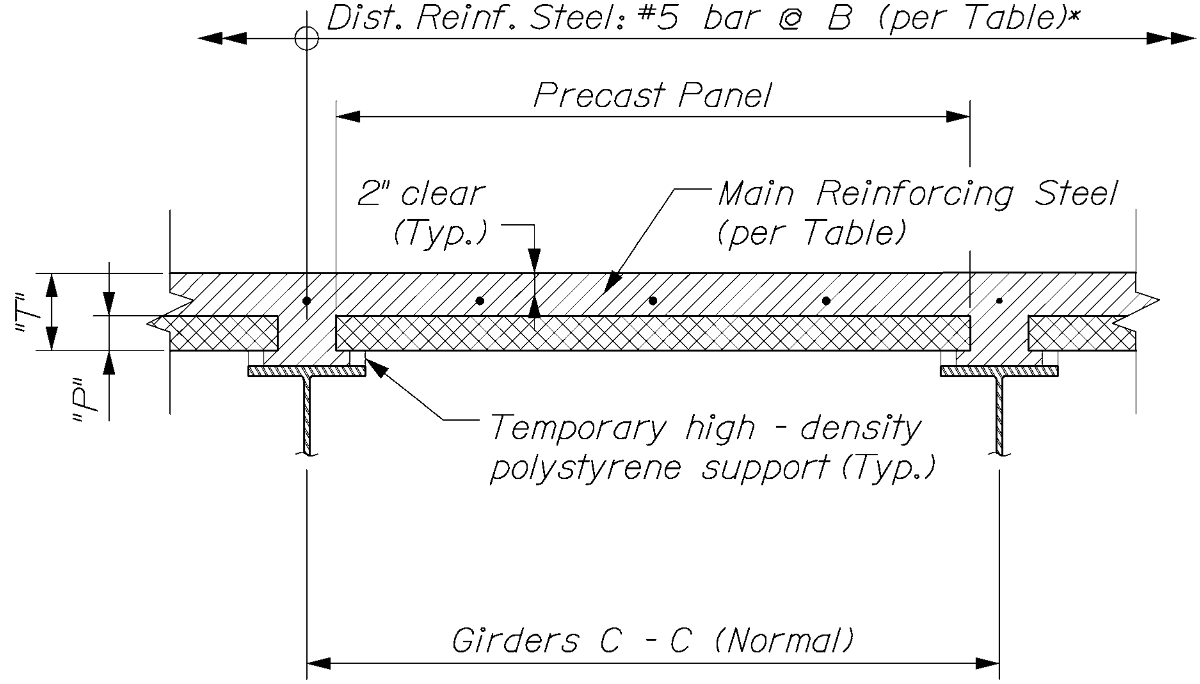
### 6.2.2.3 Precast Deck Panels

The Contractor may be given the option of constructing the concrete deck with precast, prestressed concrete deck panels. Standard designs of precast deck panels on steel girders are covered in this section. For structures with wider flanges and smaller girder spacings, precast deck panels should incorporate plain reinforcement along with prestressed reinforcement.

Precast, prestressed concrete deck panels on steel girders are specified in accordance with Table 6-7. Refer to Figure 6-2 for an explanation of reinforcing details. The panel type and number of reinforcing strands should be indicated on the design drawings.

Standard Details 502 (07-12) do not include details for systems with slabs thinner than 8 inches. The precast deck panel cannot be used with toppings thinner than 4-3/8" because there is insufficient cover for the topping reinforcing steel. Therefore, there is no direct precast system substitution for the standard slab designs with 7" and 7-1/2" slabs. Eight inch slabs cannot be substituted due to difficulties with profile grades/ bottom of slab grades. However, designs for the 7'-0", 7'-6", 8'-0", and 8'-6" design span slabs are included in Table 6-7 so that the Structural Designer can use these shorter spans for projects that specify solely precast, prestressed panels. Spans shorter than 7 feet cannot be designed due to the inability to develop the strength of the prestressing strand.

Precast, prestressed deck panels should not be used when skews exceed 30°.




TRANSVERSE SECTION

* For extra distribution steel in negative moment areas, see text.

DESIGN: Load & Resistance Factor Design per AASHTO LRFD- Specifications for Highway Bridges 1998 and Interim Specifications through 2002

LOADING: HL-93 Modified for Strength I (Impact = 33%)

STRESSES: Prestressing steel ~ $f'_s$ = 270,000 psi
$E_s$ = 78,500,000 psi
Reinforcing steel ~ $f_y$ = 60,000 psi
Concrete ~ $f'_c$ = 4,350 psi (Cast - in - place)
$f'_c$ = 6,000 psi (Precast)

MATERIALS: Strands ~ $\frac{3}{8}$" $\phi$, Grade 270 (Low relaxation)

**Figure 6-2 Precast Deck Panels on Girder Superstructures**

The deck panels and the main reinforcement should be normal to the girders. Superelevated and curved bridges require special consideration for variable temporary blocking thickness and non-parallel panel layouts.

**Table 6-7 Precast Deck Panels on Steel Girders**

| Panel Type | Maximum Girder Spacing | Slab "T" (in) | Panel "P" (in) | Number of Strands | | |
|---|---|---|---|---|---|---|
| | | | | Compression Flange Width | | |
| | | | | 1' | 1'-6" | 2' |
| A1 | 7'-6" | 8.0 | 3.5 | 15 | 15 | 15 |
| A2 | 8'-0" | 8.0 | 3.5 | 15 | 15 | 15 |
| A3 | 8'-6" | 8.0 | 3.5 | 17 | 16 | 16 |
| A4 | 9'-0" | 8.0 | 3.5 | 19 | 17 | 17 |
| A | 9'-6" | 8.0 | 3.5 | 21 | 19 | 18 |
| B | 10'-0" | 8.5 | 3.5 | 22 | 21 | 19 |
| C | 10'-6" | 9.0 | 3.5 | 24 | 22 | 20 |
| D | 11'-0" | 9.5 | 3.5 | 27 | 24 | 22 |
| E | 11'-6" | 10.0 | 3.5 | 30 | 27 | 25 |
| F | 12'-0" | 10.5 | 3.5 | 33 | 30 | 28 |

The Structural Designer should indicate on the design drawings the size and location of reinforcing steel for the top mat and for the cast-in-place end sections as indicated in Section A - A of Standard Detail 502 (08) Precast Concrete Deck Panels.

The appropriate notes found in Appendix D Standard Notes Superstructures should be included in the contract drawings.

### *6.2.3 Drilled and Anchored Bolts/Bars*

There are two general conditions where drilled and anchored bolts or reinforcing steel will be used. The first is where adequate concrete thickness is available to develop the yield strength of the anchor. The second is where adequate concrete thickness is not available to develop the yield strength of the anchor.

When adequate concrete thickness is available, anchorage will be designed for the yield strength of the anchor. The unconfined pullout strength specified on the plans will equal the yield strength of the anchor (refer to Table 6-9 and Table 6-10).

When adequate concrete thickness is not available, the design capacity of the anchor will be limited by the unconfined pullout strength of the concrete (refer to Table 6-8).

The following tables may be used for the design of drilled and anchored bolts and reinforcing bars in lieu of a more precise analysis. The strengths given are ultimate strengths and therefore appropriate load factors should be applied to design loads. A concrete compressive strength of 3 ksi is assumed.

**Table 6-8 Concrete Unconfined Pullout Strength**

| **Depth (in.)** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **12** | **15** | **18** | **24** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Unconfined Pullout (kips)** | 3.5 | 7 | 11 | 17 | 24 | 32 | 40 | 50 | 70 | 120 | 170 | 300 |

**Table 6-9 Anchor Bolt Yield Strength (kips)**

| **Bolt Size (in.)** | **3/8** | **½** | **5/8** | **¾** | **7/8** | **1** | **1-1/8** | **1- ¼** | **1-3/8** | **1- ½** |
|---|---|---|---|---|---|---|---|---|---|---|
| **A449 & A325** | 6.5 | 12 | 19 | 28 | 39 | 51 | 56 | 71 | 85 | 103 |
| **A709 Grade 50** | 3.5 | 7 | 11 | 16 | 23 | 30 | 38 | 48 | 57 | 70 |
| **A709 Grade 36** | 2.5 | 5 | 8 | 12 | 16 | 21 | 27 | 34 | 41 | 50 |

**Table 6-10 Reinforcing Steel Yield Strength**

| **Bar Size (#)** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **14** | **18** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Yield (kips)** | 6.5 | 12 | 18 | 26 | 36 | 47 | 60 | 76 | 93 | 135 | 240 |

6.2.3.1 Specification Procedures

The following information should be provided on the plans when specifying drilling and anchoring.

- Anchor size
- Anchor spacing or layout
- Anchor type
- Unconfined pullout requirements
- Minimum anchor embedment depth

Anchors set by drilling and anchoring have been divided into three general types:

- Type I - Anchor bolts size one inch or greater
- Type II - Anchor bolts smaller than one inch
- Type III - Reinforcing steel anchors

A list of prequalified anchoring materials for each type of anchor is available at www.state.me.us/mdot/planning/products/anchor.htm. Appropriate notes from Appendix D Standard Notes Drilled and Anchored Bolts and Reinforcing Steel should be included on the plans.

The minimum embedment depth given on the plans is based on the depth required to achieve adequate concrete strength. Additional depth above Table 6-8 requirements may be specified, if the Structural Designer feels it is required, as the added cost of increased embedment depth is minimal. However, the embedment should not be less than shown in Table 6-8 without a more precise analysis or a proof load test.

When available concrete thickness is not adequate to provide unconfined pullout strength equal to the yield of the anchor, or the condition of the concrete is a concern, a proof load test may be specified. This can be done by including Supplemental Specification, Section 502 (Proof Load Testing) in the contract book and including the appropriate pay items.

Because of limitations of readily available testing equipment, proof load tests should not be specified for unconfined pullouts in excess of 50 kips. If an unconfined pullout test greater than 50 kips is needed, the Structural Designer should consult with MaineDOT's Transportation Research Division to determine the availability and practicability of specifying a proof load test.

A. Type I Anchors

Bearing plate anchor bolts sizes 1" and 1-1/2" are specified in the Standard Details. For other sizes of bearing anchor bolts, specify the minimum embedment depth and anchor bolt size.

For all other anchor bolts, specify the anchor bolt as a Type I anchor and include the appropriate notes found in Appendix D Standard Notes. Specify the bolt size, spacing, minimum embedment depth (from Table 6-8), and the unconfined pullout requirements.

B. Type II Anchors

For bridge rail anchors mounted on curbs with adequate concrete depth, include Special Provision Section 507 - Railings (Anchor Bolt Installation). When available embedment is less than is required in Special Provision 507, the Structural Designer should either do a more precise analysis or use other methods of attaching anchor bolts.

For all other anchor bolts, specify that the anchor is a Type II anchor and include the appropriate notes from Appendix D. Specify the bolt size, spacing, minimum embedment depth (from Table 6-8), and unconfined pullout requirements.

C. Type III Anchors

When using drilled and anchored reinforcing bars, specify that they are Type III anchors and include the appropriate notes from Appendix D. Specify rebar size, spacing, minimum embedment depth (from Table 6-8), and unconfined pullout requirements.

For concrete curb and barrier rail reinforcing steel anchors, use Table 6-11 when appropriate.

Add additional bar length to the dimensions in the reinforcing steel schedule (for embedment) according to Table 6-12, or to the maximum available embedment if less. The added bar length is to account for the fact that some products on the approved list may require embedment length greater than the minimum given on the plans, which is based on concrete strength only.

Added bar lengths must be equal to or greater than the embedment depth actually specified on the plans.

**Table 6-11 Reinforcing Steel Anchorage**

| Curbs with Steel Bridge Railing | | | |
|---|---|---|---|
| **Minimum Available Embedment Depth (in)** | **Minimum Bar Size** | **Maximum Bar Spacing (in)** | **Unconfined Pullout (k)** |
| 4 | # 5 | 6 | 4 |
| 4-1/2 | # 5 | 9 | 6 |
| 5 | # 5 | 12 | 8 |
| 6 | # 5 | 18 | 12 |
| **F-Shaped Concrete Barrier** | | | |
| 5 | # 5 | 6 | 8 |
| 5-1/2 | # 5 | 9 | 12 |
| 6 | # 5 | 12 | 16 |
| 7-1/2 | # 6 | 18 | 24 |

Notes:

1. Minimum available embedment depth is defined as slab thickness minus 2”.
2. Curb requirements for steel bridge rail assume a 1’-5” minimum concrete curb width.
3. Concrete barrier requirements assume either 32” or 42” barrier height.

**Table 6-12 Additional Bar Length**

| Bar Size | Bar Length |
|---|---|
| #4 & #5 | 12” |
| #6 & #7 | 15” |
| #8 & #9 | 18” |

## References

AASHTO, 1998 and Interims, *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, Washington, DC

Oesterle, R., 1989, Design of Precast Prestressed Bridge Girders Made Continuous, *NCHRP Report 322*

Precast Concrete Institute, 1997, *PCI Precast Prestressed Concrete Bridge Design Manual*, USA

Precast Concrete Institute New England Region Technical Committee, *Design Guidelines for Post-Tensioning and Splicing the NEBT Girder*