CHAPTER 5 – CONCRETE STRUCTURES

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5.4─MATERIAL PROPERTIES

5.4.1─General

The following shall replace *A5.4.1*.

All structural concrete classes and associated material properties shall conform to Section 901 of *Standard Specifications*. Design strength for each class shall be as specified in Structural Concrete Classes Summary Table, included herein.

Structural elements (except drilled shafts) meeting mass concrete definition in Section 901 of *Standard Specifications* shall be identified as mass concrete in the contract plans and quantities shall be included in the pay item for Class MASS Concrete.

C5.4.1

The following shall replace *AC5.4.1*.

All materials and tests shall conform to the latest edition of *LADOTD Standard Specifications for Roads and Bridges (Standard Specifications)*. 2016 Edition of *Standard Specifications* introduced the surface resistivity requirement for all structural concrete classes. This requirement is intended to provide adequate corrosion protection for structural concrete.

Structural Concrete Classes Summary Table

(In conformance with 2016 Standard Specifications)

Structural Concrete Classes	Design Strength f'c (psi) [1]	Applications	Mix Design Requirements	Surface Resistivity Requirement
A1	4,000	All structural elements except the ones identified for other concrete classes[4]	Per Master Proportion Table 901-3 of 2016 Standard Specifications	Per Table 901-3 and 901-6 of 2016 Standard Specifications
A2	6,000			
A3	8,500			
MASS (A1)	4,000	Structural Elements with a least dimension of 48 inches or greater (Except for drilled shafts)[4]		
MASS (A2)	6,000			
MASS (A3)	8,500			
S	4,000	Drilled Shafts and Seals		
P1	6,000 at Final, 4,500 at Release[3]	Precast-Prestressed Concrete Piles		
P2[2]	8,500 at Final, 6,500 at Release[3]	Precast-Prestressed Concrete Girders		
P3[2]	10,000 at Final, 7,500 at Release[3]			

1. For cast-in-place concrete Classes A1, A2, A3, MASS(A1), MASS(A2), MASS(A3) and S, the design strength, f_c' is the minimum acceptance strength for "50% pay or remove and replace" as shown in Table 901-4 of 2016 Standard Specifications. The "Average Compressive Strength at 28 days" shown in Master Proportion Table 901-3 of 2016 Standard Specifications shall not be used for design.

2. Class P2 is the standard concrete class for all precast-prestressed girders. Using Class P3 requires approval from the Bridge Design Engineer Administrator.

3. Release strength shown is the recommended value. Release strength shall be limited between 0.6 f_c' and 0.8 f_c'.

4. Class A1 and MASS(A1) are the standard concrete classes for all structural elements except the ones identified for other classes. A2, A3, MASS(A2) and MASS(A3) should be used for special applications where demanding higher strength, such as closure pour between segments in precast segmental bridges, large cantilever caps, etc.

5.4.2─Normal Weight and Structural Lightweight Concrete

5.4.2.1─Compressive Strength

The following shall supplement *A5.4.2.1*.

See D5.4.1 for structural concrete classes and design strengths.

Lightweight concrete shall not be used without prior review and approval of the Bridge Design Engineer Administrator.

C5.4.2.1

The following shall replace *AC5.4.2.1*.

All materials and tests shall conform to *LADOTD Standard Specifications for Roads and Bridges*.

5.4.2.3─Shrinkage and Creep

The following shall supplement *A5.4.2.3*.

See *D3.12.5* for additional provisions for estimating movements due to shrinkage and creep.

5.4.3─Reinforcing Steel

5.4.3.1─General

The following shall replace the first paragraph of *A5.4.3.1*.

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards specified in *Standard Specifications*, or as amended by Supplemental Specifications and/or Project Special Provisions.

Steel materials shall be Grade 60 in accordance with *Standard Specification*. Grade 75 reinforcing steel is allowed for common use in Welded Wire Fabric (WWF). Grade 40 reinforcing steel shall not be used without prior review and approval of the Bridge Design Engineer Administrator.

Epoxy coated reinforcing steel shall not be specified.

Galvanized, stainless, stainless clad, low-carbon chromium (ASTM A1035/A1035M), or any type other than ASTM A615 “black” reinforcing steel shall not be specified without prior review and approval of the Bridge Design Engineer Administrator.

C5.4.3.1

The following shall replace *AC5.4.3.1*.

The general policy is to use uncoated, “black” reinforcing steel with high performance concrete with low permeability and an increased concrete cover for corrosion protection.

Additional measures for corrosion protection may be specified by the Bridge Design Engineer Administrator on a case by case basis for bridges located in coastal areas that are deemed of higher importance.

5.4.4─Prestressing Steel

5.4.4.1─General

The following shall replace *A5.4.4.1*.

The preferred diameter of prestressing strands is 0.6 in.

Use of 0.5 in. diameter strands is acceptable.

The use of 0.375 in. diameter strands in the top flange of prestressed concrete girders to assist in supporting stirrups and controlling temperature shrinkage is acceptable. The use of 0.375 in. diameter strands as primary strands for special circumstances shall be approved by the Bridge Design Engineer Administrator.

Prestressing steel shall be low relaxation strand Grade 270.

Stress-relieved (normal relaxation) strands shall not be allowed.

High-strength steel bars shall be ASTM A722 Type 1 (Plain) or Type 2 (Deformed) Grade 150.

5.4.5─Post-Tensioning Anchorages and Couplers

The following shall supplement *A5.4.5.*

Use of strand couplers is not allowed.

5.4.6─Ducts

The following shall supplement *A5.4.6.*

Special provisions for ducts shall be prepared in accordance with industry best practices and recommendations from PTI, ASBI, FHWA and other applicable research. Special provisions shall be reviewed and approved by the Bridge Design Engineer Administrator.

C5.4.6

Applicable industry research publications:

- *PTI M55.1-03—Specification for Grouting of Post-Tensioned Structures*, second edition; Post-Tensioning Institute, Farmington Hills, MI. April 2003.
- *Federal Highway Administration—Post-Tensioning Tendon Installation and Grouting Manual*, Washington DC, May 2004.
- *VSL International LTD.—Grouting of Post - Tensioning Tendons*, Lyssach, Switzerland, May 2002.

5.5.4.2─Resistance Factors

5.5.4.2.2─ Segmental Construction

The following shall supplement *A5.5.4.2.2.*

Unbonded post-tensioning systems are not allowed unless approved by the Bridge Design Engineer Administrator for special cases.

5.6─DESIGN CONSIDERATIONS

5.6.3─Strut-and-Tie Model

5.6.3.1─Strut-and-Tie Model

C5.6.3.1

The following shall supplement *AC5.6.3.1.*

Typical situations in which the strut-and-tie model should be used for the design of elements include hammerhead pier cap cantilevers, near the supports of pile and column bent caps, and shear-to-bearing load transfer at the ends of prestressed and reinforced concrete beams.

5.7─DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.3─Flexural Members

5.7.3.4─Control of Cracking by Distribution of Reinforcement

The following shall supplement *A5.7.3.4.*

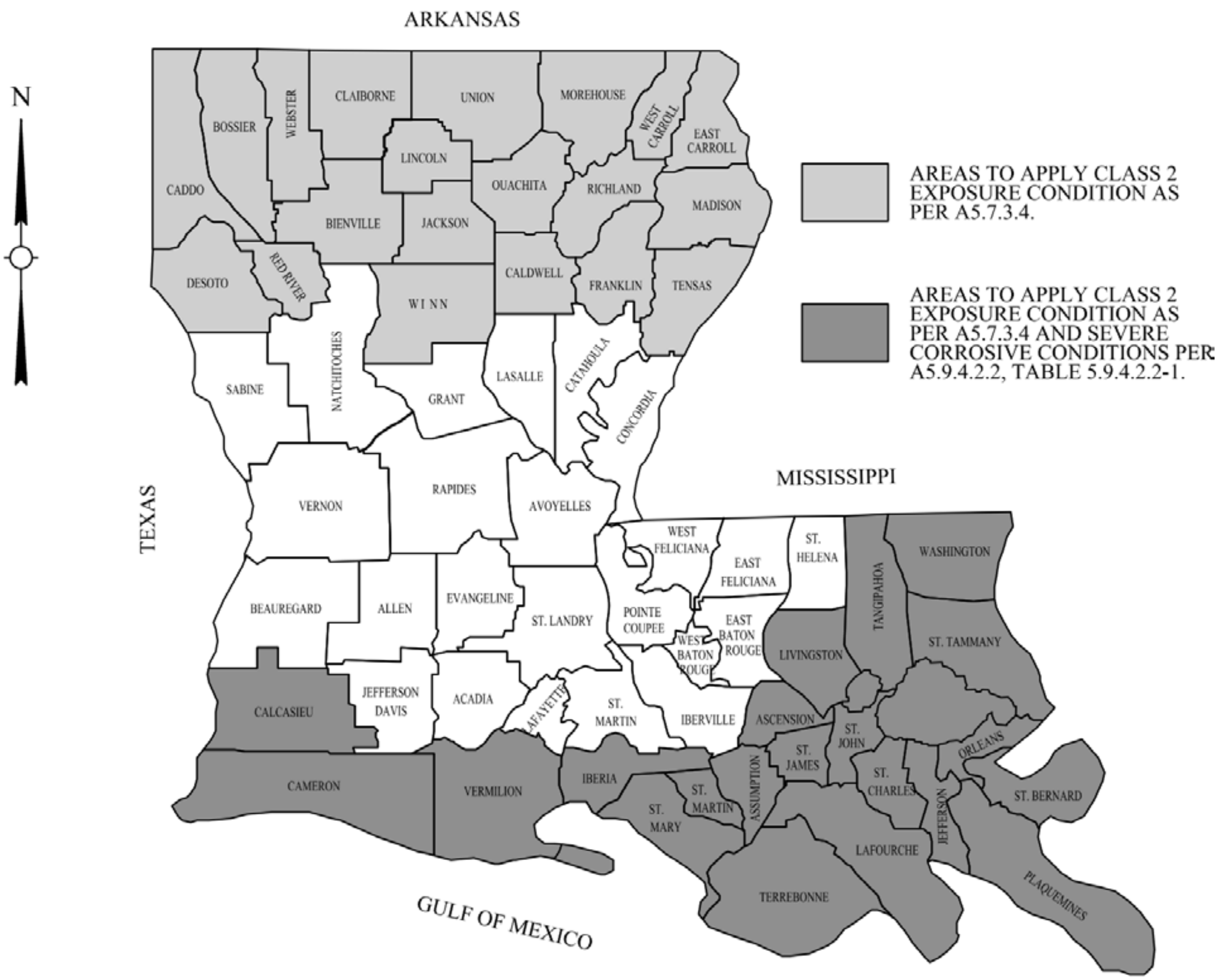
The maximum value of d_c shall be taken as 2.0 inches plus the radius of flexural reinforcement closet to the tension fiber.

Class 2 exposure conditions shall be applied to structures located in the coastal zone and in northern areas where deicing salt is frequently used. The environment exposure map below identifies areas where Class 2 exposure condition shall be applied. Class 1 exposure shall be applied to all other areas.

C5.7.3.4

The following shall supplement *AC5.7.3.4.*

For the value of d_c, all concrete cover greater than 2 inches is "sacrificial" concrete that is provided to address durability under severely corrosive exposure conditions or for wearing surface.



<u>Environmental Exposure Map</u>

5.7.3.5─Moment Redistribution

The following shall replace *A5.7.3.5.*

Redistribution of negative moments is not allowed as stated in *D4.6.4.*

C5.7.3.5

The following shall replace *AC5.7.3.5.*

The result of such redistribution is not significant, but the methodology can result in unintentional miscalculation of negative and positive moment requirements.

5.7.3.6─Deformations

5.7.3.6.2─Deflection and Camber

The following shall supplement *A5.7.3.6.2.*

The terms related to prestressed concrete girders deflection and camber (including terms for design data and field measured data) are defined as follows:

C5.7.3.6.2

The following shall supplement *AC5.7.3.6.2.*

Accurate predictions of camber and deflection are difficult due to many factors including, but not limited to:

Terms for Design Data:

- C_1 – estimated initial girder camber due to prestress force and girder self-weight at transfer
- C_2 – estimated girder camber at erection
- C_3 – estimated final girder camber
- D_1 – upward deflection due to prestress force at transfer
- D_{2a} – downward deflection due to girder self-weight at transfer
- D_{2b} – downward deflection due to girder self-weight at erection
- D_3 – downward deflection due to non-composite dead load including deck, diaphragms, and haunch
- D_4 – downward deflection due to composite dead load of barrier weight
- D_5 – total dead load downward deflection $= D_3 + D_4$

Terms for Field Measured Data:

- MC_1 – girder camber measured at 18 hours after release
- MC_{1a} – girder camber measured at 28 days
- MC_2 – girder camber measured at 21 days before riser pour
- f_{b1} – compressive concrete break strength measured at 18 hours after release
- f_{b1a} – compressive concrete break strength measured at 28 days
- f_{b2} – compressive concrete break strength measured at 21 days before riser pour
- E_{b1} – concrete modulus of elasticity measured at 18 hours after release
- E_{b1a} – concrete modulus of elasticity measured at 28 days
- E_{b2} – concrete modulus of elasticity measured at 21 days before riser pour

Deflection and camber in prestressed concrete girders shall be computed per PCI multiplier method as follows:

Step 1: Determine upward deflection (D_1) due to prestress force at transfer. The modulus of elasticity of concrete at transfer and the girder overall length shall be used.

Step 2: Determine downward deflection at transfer (D_{2a}) and at erection (D_{2b}) due to girder self-weight. The modulus of elasticity of concrete at transfer (E_{ci})

- Material property changes with time such as the modulus of elasticity of concrete.
- Creep and shrinkage, which are affected by environmental conditions such as ambient relative humidity and temperature. Creep of the concrete is primarily responsible for the camber growth.
- Lifting, handling, storage and shipping of girders typically contribute to camber growth, due to instantaneous loss of member weight due to "bouncing". Also the position of support points during storage and shipping being located too far from the girder ends will increase inelastic instantaneous creep of the extreme bottom fiber, resulting in even more camber growth than anticipated.

As discussed in *PCI Bridge Design Manual*, there are three methods for estimating long term camber and deflections. These methods are listed in order of increasing complexity and accuracy:

- multiplier methods
- improved multiplier methods, based on estimates of prestress loss
- detailed analytical methods

For practical purposes, the PCI multiplier method is adopted to calculate the estimated camber at erection provided that initial camber due to prestress and deflection due to girder self-weight at transfer are calculated separately. The use of PCI multipliers has shown to give reasonable estimates for camber at the time of erection. Refer to *PCI Bridge Design Manual* for more discussion on camber prediction.

Actual girder camber at time of casting the deck slab being equal to 2 to 3 times the initial girder camber at release is not uncommon. The amount varies based upon girder type, magnitude of prestress force, age of girder and storage and shipping methods.

For prestressed girder projects in which the contractor elects to fabricate all the girders at the same time but girder placement will extend months after casting (such as for phased construction or very large projects), the contractor must be responsible for camber growth.

shall be used. Use the girder overall length when calculating D_{2a}. Use the girder design span (center to center of bearings) when calculating D_{2b}.

Step 3: Determine downward deflection (D_3) due to non-composite dead load including deck, diaphragms and haunch weight. Use the modulus of elasticity at service loads (E_c) and girder design span.

Step 4: Determine downward deflection (D_4) due to composite dead load of barrier weight. Future wearing surface shall not be included. Use the modulus of elasticity at service loads (E_c) and girder design span.

Step 5: Determine the total dead load downward deflection (D_5).

$$D_5 = D_3 + D_4$$

Step 6: Determine estimated initial upward camber (C_1) due to prestress and self-weight at transfer.

$$C_1 = D_1 - D_{2a}$$

Step 7: Determine estimated upward camber at erection (C_2).

$$C_2 = 1.80D_1 - 1.85D_{2b}$$

Step 8: Determine final upward girder camber (C_3).

$$C_3 = C_2 – D_5$$

The final girder camber (C_3) shall be a minimum 1/2" upward and shall not exceed the haunch thickness (as specified in *D.5.14.1.2*) minus 1/2". In addition, minimum haunch thickness at any location along girder line and across girder top flange shall be 1/2". These limits give the contractor 1/2" allowance to ensure that the final girder camber will not sag and that the top of the girder will not encroach the deck. The minimum haunch thickness usually occurs at the center of span and the edge of girder top flange at the lower side of deck cross slope. Deck cross slope, vertical geometry, and super-elevation must be considered when determining the required haunch thickness. Refer to D5.14.1.2 for additional haunch requirements.

The Camber Data Table shown below shall be included in contract plan.

Camber Data Table

CAMBER DATA TABLE																
SPAN NO.	GIRDER DESIGNATION	DESIGN DATA				FIELD MEASURED DATA *									* DATE OF GIRDER CASTING	* DATE OF RISER POUR
		C1 (IN.)	C2 (IN.)	C3 (IN.)	D5 (IN.)	MC1 (IN.)	MC1a (IN.)	MC2 (IN.)	fb1 (KSI)	fb1a (KSI)	fb2 (KSI)	Eb1 (KSI)	Eb1a (KSI)	Eb2 (KSI)		
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

The camber design data (C_1, C_2, C_3 and D_5) are provided by the EOR. The field measured data (MC_1, MC_{1a}, MC_2, f_{b1}, f_{b1a}, f_{b2}, E_{b1}, E_{b1a} and E_{b2}) and the dates of girder casting and riser pour shall be recorded by the contractor in the Camber Data Table. The Camber Data Table shall be submitted to the EOR for review at least 14 days prior to riser pour. When field measured MC_1 or MC_2 differ more than 1/2" (+ or -) from the estimated "C_1" or "C_2", the contractor shall notify the EOR immediately to investigate corrective measures, such as modify risers and/or roadway profile, etc.

Refer to LG Girder Standard Plans for camber details.

5.8─SHEAR AND TORSION

5.8.2─General Requirements

5.8.2.7─Maximum Spacing of Transverse Reinforcement

The following shall supplement *A5.8.2.7.*

For all concrete girders that span existing or future traffic lanes or railroad tracks, transverse reinforcement, including lower flange confinement reinforcement, is required throughout the full length of the girder and the spacing shall not exceed 12 inches.

Stirrups in all bent caps shall comply with the following spacing requirements:

C5.8.2.7

The following shall supplement *AC5.8.2.7.*

This design requirement is intended to contain damaged concrete, following vehicle or train collisions, and to prevent spalled concrete from falling on vehicles or trains. This will also provide minimum shear strength to better facilitate temporary shoring following collisions, which may better enable a damaged structure to carry traffic until such time as the structure can

- The first stirrup shall be placed no more than 3 inches clear from the face of a pile, drilled shaft, column, or the edge of the cap.
- The spacing between the first stirrup and an adjacent stirrup shall not exceed 6 inches.
- The spacing between all remaining stirrups shall not exceed 12 inches.

5.8.3.4─Procedures for Determining Shear Resistance

The following shall supplement *A5.8.3.4.*

A5.8.3.4.1 shall be used for reinforced concrete sections.

A5.8.3.4.2 shall be used for prestressed concrete sections and reinforced concrete sections that are not covered by *A5.8.3.4.1*.

The same method used for the design shall also be used for the as-designed bridge rating calculations.

5.9─PRESTRESSING AND PARTIAL PRESTRESSING

5.9.4─Stress Limits for Concrete

5.9.4.2─For Stresses at Service Limit State after Losses-Fully Prestressed Components

5.9.4.2.2─Tension Stresses

The following shall supplement *A5.9.4.2.2*.

Severe corrosive conditions referenced in *Table A5.9.4.2.2-1* shall be applied to all structures located in coastal zones as identified on environment exposure map in *D5.7.3.4*. Moderate corrosive conditions shall be applied to all other areas.

For structures located on routes with current ADTT greater than 2500, tensile stress limit in prestressed concrete at service limit state shall be limited to $0.0948\sqrt{f_c'}$.

5.10—DETAILS OF REINFORCEMENT

5.10.3─Spacing of Reinforcement

5.10.3.3─Minimum Spacing of Prestressing

be repaired.

Tendons and Ducts

5.10.3.3.1—Pretensioning Strand

The following shall replace *A5.10.3.3.1*.

Strands shall be spaced no less than 2 inches' center-to-center regardless of strand size.

Prestressing strands shall be distributed evenly across a row to achieve uniform pretensioning in the girder end zones. Clustering of strands in the bottom corners of beams should be avoided.

Prestressing strands shall not be bundled to touch one another.

C5.10.3.3.1

The following shall replace *AC5.10.3.3.1*.

The 2.00-inch center-to-center spacing has been in effect for many years with successful use by many states, including Louisiana.

5.10.10─Pretensioned Anchorage Zones

5.10.10.1─Splitting Resistance

The following shall supplement *A5.10.10.1*.

For pretensioned girders with depth < 4 feet, A_s shall be taken as the total area of the vertical reinforcement located within a distance of 12 inches from the end of the member.

C5.10.10.1

The following shall supplement *AC5.10.10.1*.

For pretensioned girders with depth < 4 feet, placing the required splitting resistance reinforcement over a length of H/4 will result in reinforcement congestion and possible voids due to poor consolidation at the end zones.

5.10.11—Provisions for Seismic Design

5.10.11.1—General

The following shall supplement *A5.10.11.1*.

Lateral restraint shall be designed to resist seismic forces per the seismic zone classification map in *D3.10* and the requirements of *A5.10.11.2* and *A5.10.11.3*.

For all prestressed concrete girder bridges, provide concrete seismic shear keys cast integral with the pier cap to resist applied seismic forces. Refer to Part III Chapter 1 – LG Girder design aids for shear key details.

Sufficient bearing surface between the shear keys and girder bottom flanges shall be provided to transfer the seismic force.

C5.10.11.1

The following shall supplement *AC5.10.11.1*.

Concrete seismic shear keys replace the past practice of using clip angles and anchor bolts at the end of girders, which has shown poor field performance due to common misplacement of anchor bolts that caused bending of anchor bolts and corrosion of angles and anchor bolts in coastal environments.

5.11─DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.4─Development of Prestressing Strand

5.11.4.3─Partially Debonded Strands

The following shall replace *A5.11.4.3*.

Debonding of prestressed strands is not allowed without prior review and approval by the Bridge Design Engineer Administrator. If debonding is deemed necessary and approved, then the criteria of *A5.11.4.3* shall apply.

Straight strand patterns shall be used whenever possible. Combination of straight and draped/harped strand pattern may be utilized to satisfy the allowable stresses at release. The tie down points for draped/harped strands varies, but shall be consistent within a project whenever possible. The maximum uplift at each strand hold down device is typically 40 kips. If uplift force exceeds 40^k, EOR shall reevaluate the design or provide fabricator the uplift force and require fabricator use multiple hold down devices.

5.12─DURABILITY

5.12.1─General

The following shall supplement *A5.12.1.*

LADOTD's strategy to provide durability for concrete structures consists of a combination of methods: utilizing high performance concrete with permeability/surface resistivity requirements for all structural concrete elements, providing minimum concrete covers, controlling crack width by distribution of reinforcement, specifying water curing procedures in *Standard Specifications*, and providing protective measures and details as specified in *A2.5.2.1* and *D2.5.2.1*.

5.12.3─Concrete Cover

The following shall replace *A5.12.3.*

Concrete Cover for unprotected prestressing and reinforcing steel, which is defined as the distance from the edge of concrete to edge of the nearest reinforcement, shall not be less than that specified in the table below:

Concrete Cover Table

Application	Cover (inches)
All superstructure components unless otherwise specified	2
All substructure components unless otherwise specified	3
Deck top surfaces for spans in fixed bridges, Slab Span / Approach Slab top surfaces	2.5
Deck top surfaces for spans in movable bridges	1.5
Deck/Slab Span bottom surfaces	1.5
Approach Slab bottom surfaces	2
Barrier Railing, Diaphragms	1.5
Top flange and web of precast prestressed girders	1.5
All internal surfaces (not exposed to environment)	1.5
Drilled Shafts greater than or equal to 30” in diameter	6
Drilled Shafts less than 30” in diameter	3
Surfaces cast against earth	3
Roadway Median Barrier, walls, shear keys, risers	2

5.12.4─Protective Coatings

The following shall replace *A5.12.4.*

Refer to *D5.4.3.1* for the policy on protective coatings.

5.13─SPECIFIC MEMBERS

5.13.2─Diaphragms, Deep Beams, Brackets, Corbels and Beam Ledges

5.13.2.2─Diaphragms

The following shall supplement *A5.13.2.2.*

Intermediate diaphragms (ID) for precast prestressed concrete girder spans shall be provided as specified in the Policy for Intermediate Diaphragms table. End diaphragms (ED) shall be provided for all precast prestressed concrete girder spans. Both ED and ID shall have a minimum width of 8 inches and shall extend full depth from the bottom of deck to top of girder's bottom flange.

C5.13.2.2

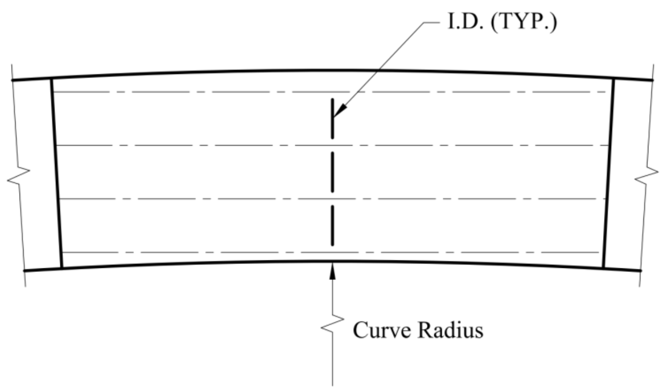
The following shall supplement *AC5.13.2.2.*

The study report for intermediate diaphragms is included in *BDEM Part IV*.

Refer to BDEM Part III Chapter 1 - LG girder for typical ID and ED details.

Policy for Intermediate Diaphragms

Situations	Requirement for Intermediate Diaphragms (ID)
All spans unless otherwise specified as follows:	ID is not required.
Case 1: Spans over roadways, railroads, navigational channels, and water body with anticipated marine traffic under normal loading condition except for Cases 2 and 3	One ID shall be provided at center of span.
Case 2: Spans on curve with curved girders only	Requirement of ID shall be determined for the design condition. Minimum one ID shall be provided.
Case 3: Spans subject to wave force, extreme high wind conditions, other anticipated lateral forces, or other unusual loading conditions	Requirement of ID shall be determined for the design condition. Minimum one ID shall be provided.



Spans on Curve

5.13.4—Concrete Piles

5.13.4.4—Precast Prestressed Piles

The following shall supplement *A5.13.4.4.*

Refer to LADOTD Bridge Design Standard Plans for precast prestressed concrete pile details.

5.14—PROVISIONS FOR STRUCTURE TYPES

5.14.1—Beams and Girders

5.14.1.2—Precast Beams

The following shall supplement *A5.14.1.2*.

Louisiana Girder (LG) types, LG-25, LG-36, LG-

C5.14.1.2

The following shall supplement *AC5.14.1.2*.

The efficiency factor of a girder section is

45, LG-54, LG-63, LG-72, and LG-78, shall be the standard precast prestressed concrete (PPC) girders used for new construction and bridge widening. Preliminary design tables, standard bearing pads, and Standard Plans for LG girders have been developed. Refer to Part III, Chapter 1, LG Girders and LG girders Standard Plans for more information.

Quad Beam, AASHTO Type II, III, IV, BT-72 and BT-78 are allowed for bridge rehabilitation projects with the approval of the Bridge Design Engineer Administrator. AASHTO Type I and Type IV Modified girders are not allowed.

Value engineering proposals to change LG girders to other girder types are not allowed.

Maximum span lengths for PPC girders along with the associated maximum prestressing forces immediately prior to transfer are specified in the Girder Maximum Span Length Table.

Girder spacing within a bridge cross-section shall be equal where practical. The girder spacing shall not exceed 12.0 feet center-to-center for I-shaped girders.

The girder types and strand patterns shall be minimized within a project to simplify fabrication. Girders with similar length and loads shall use the same girder type and strand pattern. Refer to *D2.5.2.7.1* for requirements on exterior girder capacity.

Strand pattern details showing strand layouts, number and spacing of strands, concrete cover and edge clearances, and layout of all mild reinforcing steel shall be shown in contract plans.

All girder related design data shall be shown in a girder data table. Refer to LG girder design aids in Part III, Chapter 1 for a girder data table template.

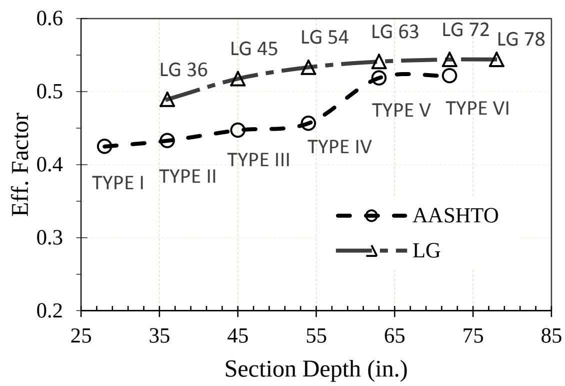
defined as:

$$\rho = \frac{I}{A\, y_b\, y_t} = \frac{r^2}{y_b\, y_t}$$

ρ = Efficiency factor of a section
I = Moment of inertia of a section
y_b = Distance of bottom fiber from centroid of section
y_t = Distance of top fiber from centroid of section

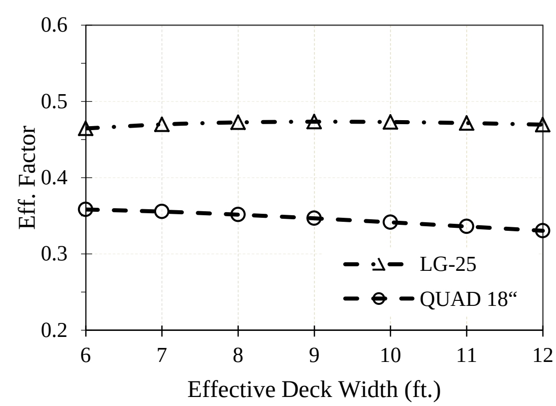
r = Radius of gyration of section = $\sqrt{\dfrac{I}{A}}$

Figure below shows efficiency factor comparison of new LG girders and AASHTO type girders.



Efficiency Factor (LG vs. AASHTO)

LG-25 is more efficient than Quad Beam as shown in Figure below. Since both LG-25 and Quad Beam lack a top flange and are generally for short spans where live load dominates the design, composite sections assuming 8.5” deck thickness are used in the evaluation of girder efficiency factors.



Efficiency Factor (LG-25 vs. Quad Beam)

Square beam ends shall be used for all prestressed concrete girders bridges, except where utilizing square beam ends is not feasible, then clipped ends maybe used. Embedded plate shall be provided at girder ends for LG-45 to LG-78. Refer to LG girder Standard Plans for details.

Utilizing square beam ends simplifies both the girder production and construction. Providing embedded plate at long girder ends prevents or minimizes end zone cracking.

The haunch thickness at girder bearing centerline shall be minimum 2 inches for spans less than 90 feet, 3 inches for spans from 90 to 120 feet, and 4 inches for spans greater than 120 feet. Haunch thickness shall be included in weight calculation, but shall be omitted in the calculation of composite section properties used in determining live load effect.

Refer to D5.7.3.6.2 for additional haunch thickness and camber requirements that must be considered when determining haunch thickness. Reinforcement shall be provided in haunches exceeding 4 inches in thickness. Girder haunch shall not exceed 6 inches at any location.

Designers shall pay special attention to the haunch thickness of prestressed girders when used in conjunction with a high degree of vertical and horizontal curvature (super-elevation) which could present challenges to meeting haunch dimension requirements.

For riser policy refer to Part II, Vol 1, Chapter 14 – Joints and Bearings.

PPC girders shall not be used in a curved bridge where the offset between an arc and its chord exceeds 1 foot. Refer to *PCI Bridge Design Manual* for additional design considerations for skewed and curved bridges.

The notes below shall be included in PPC girder detail sheets or general notes sheets for all projects.

"The contractor is responsible for stability of precast prestressed concrete girders during fabrication, storage, transportation, erection, and deck placement. Supporting analysis and calculations stamped, signed, and dated by a Louisiana licensed professional engineer and shop drawings showing the method of lifting the girder, lifting locations and details, support (dunnage) locations for storage and transportation details, and erection bracing details shall be submitted to the EOR for review.

Any inherent stability provided by cast-in-place diaphragms shall not be considered by the contractor in designing the required construction bracing. The diaphragms are provided to restrain lateral movement

Girder stability during each phase of construction is dependent on the type of lifting equipment and pick up methods and therefore, is the responsibility of the contractor.

For extremely long girders (typically > 160 feet), the contractor may consider using lifting brackets instead of using lifting loops; so that the girder would be lifted from below its center of gravity. The brackets may eliminate the chance of an "off center" lifting which may occur when using lifting loop on the top flange.

of girders when the bridge is in-service and are not intended or allowed for use as construction stability bracing."

During the design process, the EOR shall ensure that all girders, while within the allowable stress limits, can be supported on dunnage within 3.0 feet from their ends or as calculated.

The EOR shall determine whether the girder can be picked up in accordance with the lateral stability requirements in the *PCI Bridge Design Manual*; however, the pick-up point locations shall not be shown on the contract plans.

During the construction phase, the EOR shall verify that contractor shop drawings and supporting calculations for girder storage, lifting, and handling meet the most current lateral stability analysis procedure provided in the *PCI Bridge Design Manual* to ensure that the proposed girder stability could be achieved within the allowable stress limits listed in the contract plans.

For lateral stability examples refer to the PCI Report No. CB-02-16 titled *"Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*."

Girder Maximum Span Length Table

Girder Type	Maximum Span Length (ft.)	Maximum Prestressing Force Immediately Prior to Transfer (kip)	Maximum No. of Strands (Assume 0.6 in. Dia., 270 ksi, Low Relaxation Strands)
LG-25	53	1,408	32
LG-36	98	2,112	48
LG-45	119	2,376	54
LG-54	133	2,464	56
LG-63	154	2,816	64
LG-72	171	3,080	70
LG-78	183	3,344	76
Quad Beam (18.0 in.)	40	704	16
AASHTO Type II (36 in.)	55	750	18
AASHTO Type III (45 in.)	85	1,000	22
AASHTO Type IV (54 in.)	105	1,500	34
BT-72	125	1,850	42
BT-78	140	2,200	50

5.14.1.4─Bridges Composed of Simple Span Precast Girders Made Continuous

The following shall supplement *A5.14.1.4.*

Bridges composed of simple span precast prestressed girders made continuous utilizing positive moment connections are not allowed, due to unsatisfactory past performance of such details.

Past practice of making simple span precast girders continuous by means of continuity diaphragms shall be replaced by the new link slab method as described below for new bridges.

In the new link slab method, all precast prestressed girder spans shall be designed as simply supported spans. To minimize expansion joints, the

C5.14.1.4

The following shall supplement *AC5.14.1.4*.

Past LADOTD practice required expansion and fixed bearings at girder ends. In some earlier projects, several fixed bearings were provided in a multi-span continuous unit, which is not the best practice. Fixity was achieved by tying the end diaphragms or continuity diaphragms to the bent cap by rigid connections. The restrain and stress concentration in these rigid connections are possible causes of the observed cracking in deck, diaphragms and girder ends.

Implementation of new link slab and floating span concepts are based on extensive research on national best practices and results deck over interior supports shall be made continuous to link simply supported spans to a multi-span continuous deck unit to the maximum practical length. Continuous deck over the interior support is defined as "link slab", which is essentially a portion of continuous deck connecting adjacent simple spans. Supplemental longitudinal reinforcing (#6 @10 feet long) shall be placed on top and bottom of deck/link slab over interior supports between regular deck reinforcement for crack control purposes. Refer to BDEM Part III Chapter 1 – LG girder for typical link slab details.

The maximum multi-span continuous deck unit length shall be determined based on bridge conditions and design criteria, and shall not be greater than 750 feet.

In addition, the “floating span" concept, which requires no fixed bearing, shall be applied to simple span and multi-span continuous deck unit except those subject to extreme lateral and uplifting forces, such as wave action. In the floating span concept, girder ends are supported on expansion bearings and all loads are transferred to substructures through expansion bearing pads. The horizontal force due to shear deformation of the bearing pads shall be taken into account in substructure design. The flexibility of the substructure shall also be considered when determining the horizontal force due to shear deformation of bearing pads. For floating one span unit, assume midpoint of the span as the zero movement point. For floating multi-span continuous deck unit using link slabs, assume midpoint of the entire continuous unit as the zero movement point. If a fixed bearing is required, then the fixed bearing shall be used as the zero movement point. Refer to Part III Chapter 1 – “LG Girder” for design examples that demonstrate bearing designs for the floating span concept.

For bridges that are subject to extreme lateral and uplift forces, such as wave action, vertical restrains of the superstructure shall be designed for the loading conditions and substructures shall be designed for the forces accordingly. Refer to *D1.1* for additional design requirements for bridges subject to coastal storms.

of a pilot LADOTD research project (LTRC 14-1), where various link slab details and floating spans have been constructed and monitored. Preliminary field observations and monitoring results have concluded that the designs with these new concepts minimize deck cracking, simplify the construction, and will improve the long-term performance of precast prestressed girder bridges.

The supplemental longitudinal reinforcement specified for link slab is based on national best practices and experimental results from the pilot project. The amount provided is also verified by an analytical analysis.

The research project will continue monitoring the performance of new link slab and floating spans for several years. Upon completion of the project, the research report will be published.

The maximum multi-span continuous deck unit length is typically limited by the practical bearing design.

Link slab method may be utilized in bridge rehabilitation projects. However all existing bridge components (including joints and bearings) shall be investigated for the new loading condition.