SECTION 5: CONCRETE STRUCTURES

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5.1 Scope

This Section contains guidelines to supplement provisions of Section 5, Concrete Structures, of the AASHTO LRFD Bridge Design Specifications. These provisions apply to the design of bridges, retaining walls, and other appurtenant highway structure components constructed of normal density concrete reinforced with steel rebars, welded wire reinforcement, prestressing strands, prestressing rebars, or prestressing wires. Concrete deck design criteria are specified in Section 9 of these guidelines.

Design level load ratings of all bridges shall be performed per The Manual for Bridge Evaluation (MBE) latest edition, while the stress limits for concrete shall conform to ADOT Bridge Design Guidelines. For new bridges the design Operating Load Rating (using HL93 live load) shall be 2.0 or more. For widening of bridges, the minimum design Operating Load Rating (using HL93 live load) shall be either the Operating Load Rating (using HL93 live load) of the existing bridge or 1.5, whichever is greater. If these provisions cannot be met, coordination and approval from the ADOT Bridge Group will be required.

5.4 Material Properties

5.4.1 General

Design should be based on the material properties cited in these guidelines. The contract documents shall specify the grades or properties of all materials to be used and shall be in conformance with the latest edition of the ADOT Standard Specifications for Road and Bridge Construction. All structural concrete shall meet or exceed ADOT Class S requirements unless noted otherwise.

5.4.2 Normal Weight and Lightweight Concrete

5.4.2.1 Compressive strength

For non-prestressed applications, concrete strength greater than 5 ksi will require ADOT Materials and Bridge Groups approval. Lightweight concrete shall not be used as a structural material for bridge components unless approved by ADOT Materials and Bridge Group. Normal weight non-prestressed concrete shall have minimum compressive strengths, f´c, at 28 days as follows:

Components	f´c (ksi) min
Decks except barriers	4.5 ksi
Bridge concrete barriers	4.0 ksi
Substructures (abutments, piers, foundations and drilled shafts)	3.5 ksi
All other Class ‘S’ Concrete	3.0 ksi

Prestressed Precast Concrete shall have a specified minimum initial and final concrete strengths as specified in the table below. Concrete strengths greater than shown in that table may be used when required by design and approved by ADOT Materials and Bridge Groups.

	Initial	Final
Minimum	f´ci = 4.0 ksi	f´c = 5.0 ksi
Maximum	f´ci = 7.5 ksi	f´c = 9.0 ksi

Concrete for cast-in-place post-tensioned box girder bridges shall have initial and final concrete strengths as specified in the table below. Concrete strengths greater than the maximum limit may only be used when required by design and approved by ADOT Materials and Bridge Groups.

	Initial	Final
Minimum	f´ci = 3.5 ksi	f´c = 4.5 ksi
Maximum	-	f´c = 5.0 ksi

Final maximum compressive stress f´c of up to 6.0 ksi for cast-in-place post-tensioned box girder bridges may be specified for projects in the Phoenix and Tucson metropolitan areas. For other areas of the state, 6.0 ksi strength shall not be used without approval from ADOT Materials and Bridge Groups.

5.4.2.3 Creep and Shrinkage

Creep and shrinkage of concrete, relaxation of prestressing steel, and shrinkage of deck composite cross section are time dependent events.

In determining the effects of creep and shrinkage on the loss of prestressing force, the average annual ambient relative humidity, H shall be 40%, including whenever applying both approximate and refined estimates of prestress loss calculations in AASHTO LRFD Articles 5.9.3.3 and 5.9.3.4 respectively.

In determining the effects of creep and shrinkage on the loss of prestressing force using the refined estimate of time dependent losses in Article 5.9.3.4, the following timelines of the various stages of construction of precast pretensioned girders made composite with a deck topping may be assumed:

Time at Transfer = 1 day
Time at Deck Placement = 60 days
Time at Barrier Placement = 90 days
Time at Future Wearing Surface = 3650 days
Final Time of Superstructure (Service Life) = 18,250 days

5.4.2.4 Modulus of Elasticity

Based on an assumed unit weight of concrete of wc = 0.145 kcf, the modulus of elasticity, Ec, in ksi, shall be assumed to be:

$$E_c = 1820\sqrt{f_c^{'}}\,(\text{ksi})$$

where:

f'_c = the specified compressive strength of concrete (ksi)

For dead load calculations, the unit weight of structural concrete shall be wc = 0.150 kcf to account for steel reinforcement within the concrete.

See AASHTO section 5.4.2.4 for formulas specific to lightweight concrete, high strength concrete, and correction factors for source aggregate.

5.4.3 Reinforcing Steel

All reinforcing steel shall be supplied as Grade 60 and shall be deformed bars conforming to ASTM A615/A615M-96a, except for smooth wire spiral ties. Spiral ties shall be cold drawn wire conforming to ASTM A82 (AASHTO M32).

Welded wire reinforcing shall only be used in slope paving and prefabricated panels used for sound walls. Welded wire reinforcing shall be deformed and shall conform to ASTM A496 and ASTM A497.

Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A706 shall be specified.

For all new bridge construction or bridge deck replacement located above an elevation of 4,000 feet, epoxy reinforcement shall be specified. For bridges below 4000 feet, the design engineer shall verify with the District if de-icing chemicals are used or if corrosion instances are present on the route and location. For such instances, epoxy reinforcement shall be specified.

When bridges require epoxy coated reinforcement, all reinforcement in deck slabs, barriers, anchor slabs and approach slabs shall be epoxy coated. Any portions of reinforcement projecting into the deck slabs and approach slabs, such as girder or web stirrups, and backwall reinforcement shall also be epoxy coated.

5.4.4 Prestressing Steel

Prestressing steel for precast prestressed members and cast-in-place post-tensioned members shall be low-relaxation type and have the properties as defined in AASHTO LRFD Article 5.4.4.1 and AASHTO LRFD Table 5.4.4.1-1. For most applications, 0.5-inch or 0.6-inch diameter strands shall be specified for prestressed members. For cast-in-place post-tensioned bridge girders, 0.6-inch diameter strands may be used.

The yield point stress of prestressing steel, f_{py} shall be equal to 0.90 f_{pu} for low relaxation strands, as shown in AASHTO LRFD Table 5.4.4.1-1.

5.5 Limit States And Design Methodologies

5.5.4 Strength Limit States

5.5.4.2 Resistance Factors

For tension-controlled sections of post-tensioned box girder bridges, the resistance factor shall be taken as $\varphi = 0.95$. AASHTO LRFD Article 5.5.4.2 shall be used for all other cases.

5.6 Design For Flexural And Axial Force Effects - B Regions

5.6.2 Assumptions for Strength and Extreme Event Limit States

5.6.2.1 General

Unbonded tendons shall not be used.

5.6.3 Flexural Members

5.6.3.2 Flexural Resistance

In determining the flexural resistance for post-tensioned box girders, neither the temperature and shrinkage reinforcement, nor the distribution reinforcement shall be used. In determining the positive flexural resistance, the longitudinal flange reinforcing may be used, if necessary, to reduce reinforcement congestion.

5.6.3.5 Deformations

5.6.3.5.1 General

Live load deflections shall be limited to the values shown in the following table:

Loads Carried by the Bridge	Maximum Deflection
Vehicular Load	Span / 800
Vehicular and Pedestrian Loads	Span / 1000
Pedestrian Load	Span / 1000
Vehicular Load on Cantilever Arms	Span / 300
Vehicular and Pedestrian Loads on Cantilever Arms	Span / 375
Pedestrian Load on Cantilever Arms	Span / 375

5.6.3.5.2 Deflection and Camber

Post-Tensioned Box Girder:

The instantaneous deflection shall be calculated using the dead load including barriers but not future wearing surface, a modulus of elasticity of $E_c = 1820\sqrt{f_c^{'}}$ ksi, gross sectional properties, and calculated final losses.

The final long-term deflection may be obtained by multiplying the instantaneous deflection by a factor of three if compression steel is not included in moment of inertia calculations. For simple span bridges, an additional parabolic shaped deflection with a peak equal to 3/8 inch per 100 feet should be added to the total deflection.

Camber and screed elevations shown on the plans shall be based on the final long-term deflection. Continuous bridges, with some spans significantly longer than others, may exhibit negative camber in the shorter spans. The final long-term deflection shall be set to zero in spans exhibiting negative camber.

Precast Prestressed Girders and Beams:

Deflection calculations for precast prestressed I-girders shall be based on a cumulative construction stage analysis considering transition of section properties, maturity of concrete, prestress losses, and transition of the structure from simple to continuous. Construction stages should be defined realistically and the factors mentioned above should be applied appropriately according to the conditions for each construction stage. The release, initial and final deflections shall be shown on the plan sheets. Deflections shall be shown in thousandths of a foot at the tenth points of each span.

The release deflection equals the deflection the prestress girder undergoes at the time of release of strands. This includes the deflection due to dead load of the girder, elastic shortening, and relaxation of steel at transfer, The release deflection values shall be shown in the contract plans.

The initial deflection includes the deflection due to the dead load of the girder, the initial prestressing and the effects of creep and shrinkage up to the time of erection and prior to the diaphragm or deck pours. The time of erection should be assumed to be 60 days after release.

The total deflection includes the deflection due to the dead load of the deck, diaphragms, barriers, and the effects of long term creep on the composite girders. The future wearing surface shall be excluded from deflection calculations.

Minimum build-up at the edge of the top flange shall be 1 inch. This minimum build-up at the critical section will ensure that the flange of the girder will not encroach into the gross depth of the slab.

The top of the erected girders shall be surveyed in the field prior to placement of the deck forming. Encroachment into the slab of up to ½ inch will be allowed for random occurrences.

Design engineers should provide a note on the plans stating that the bridge seat elevations shall be verified by the contractor and provided to the engineer for review prior to the erection of the girders.

5.6.7 Control of Cracking by Distribution of Reinforcement

Except for columns and drilled shafts, in general, for service limit state, the allowable tensile stress in reinforcing steel, fs, shall be limited to 0.6·Fy , unless noted otherwise for specific bridge types and structural members in these guidelines. When the Strut-and-Tie model is used for design of structures and components, crack control provisions specified in AASHTO LRFD Article 5.6.3.6 shall be met.

5.7 Design For Shear And Torsion - B Regions

5.7.2 General Requirements

5.7.2.4 Types of Transverse Reinforcement

Welded wire reinforcement and anchored prestressed tendons shall not be used as transverse shear reinforcement without approval from ADOT Bridge Group.

5.7.3 Sectional Design Model

Note that N_u in AASHTO LRFD Articles 5.7.3.4.2 and 5.7.3.5 is not the prestress force but is the externally applied factored axial force, taken positive if tensile.

5.7.3.5 Longitudinal Reinforcement

AASHTO LRFD Equation 5.7.3.5-1 shall be evaluated at each section where simply supported girders are made continuous for live loads or where longitudinal reinforcement is discontinuous.

AASHTO LRFD Equation 5.7.3.5-2 shall be evaluated at the inside edge of the bearing area of simple end supports to the section of critical shear (d_v from the internal face of support, AASHTO LRFD Article 5.7.3.2). The values of V_u, V_s, V_p, and θ, calculated for the section dv from the face of the support, may be used.

5.7.4 Interface Shear Transfer – Shear Friction

For precast prestressed members, only the composite dead load and live load shall be considered when calculating interface shear.

For all precast girders, beams and slabs, the following note shall be shown on the plans: "The contact surface of the precast member receiving the deck or overlay shall be roughened to a depth of approximately ¼ inch."

Interface shear shall not be considered in the design of post-tensioned bridge members.

5.9 PRESTRESSING

5.9.1 General Design Considerations

5.9.1.3 Section Properties

Section properties shall be based on gross area of members for cast-in-place post-tensioned members. Section properties shall be based on the transformed area of bonded prestressing strand for precast prestressed members. Gross section properties shall be used for deflection calculations and live load distribution for precast prestressed members.

5.9.2 Stress Limitations

5.9.2.2 Stress Limitations for Prestressing Steel

For post-tensioned members, overstressing for a short period of time to offset seating and friction losses is permitted, but the maximum allowable jacking stress for low-relaxation strands shall be limited to $0.78 \cdot f_{pu}$.

For precast prestressed members, the maximum allowable jacking stress for low relaxation strands shall be limited to $0.75 \cdot f_{pu}$. Overstressing the prestressing steel to offset seating or relaxation before transfer losses is not permitted.

5.9.2.3 Stress Limits for Concrete

<table>
<tr><td colspan="2" rowspan="3"></td><td colspan="6">LOAD CASES</td></tr>
<tr><td rowspan="2">BEFORE TIME-DEPENDENT LOSSES</td><td colspan="4">AFTER LOSSES</td></tr>
<tr><td>DC + PRESTRESS</td><td>SERVICE LIMIT I</td><td>SERVICE LIMIT III</td><td>0.5(DW+DC + PRESTRESS) + (LL + IM)</td></tr>
<tr><td colspan="2">COMPRESSION (KSI)</td><td>$0.6 f'_{ci}$</td><td>$0.45 f'_c$</td><td>$0.6 \phi_w f'_c$</td><td>N/A</td><td>$0.4 f'_c$</td></tr>
<tr><td rowspan="2">TENSION (KSI)</td><td>Any region of a prestressed component in which prestressing causes compressive stresses and service load effects causes tensile stresses</td><td>N/A</td><td>0 (zero) for post-tensioned boxes

N/A for precast prestressed members</td><td>N/A</td><td>$0.0948\sqrt{f'_c}$ for precast prestressed members

0 (zero) for post-tensioned boxes</td><td>N/A</td></tr>
<tr><td>Other Regions</td><td>$0.0948\sqrt{f'_{ci}}$
$\leq 0.2 ksi$</td><td>N/A</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
</table>

5.9.3 Prestress Losses

5.9.3.2.2 Friction

For multi-span bridges, the cable path should have its low point at or near mid span. Design should be based on usage of galvanized rigid ducts with K = 0.0002 and μ = 0.25. However, μ value of 0.15 may be used when comparing initial force coefficient with values shown in Table 5.9.3.2.2b-1 “Friction Coefficients for Post-Tensioning Tendons”.

Value of μ should be based on experimental data for the materials specified and shall be within the values given in the “Friction Coefficients for Post-Tensioning Tendons” table. In the absence of such data, a value of μ = 0.25 may be used.

Loss Due to Anchorage Set

$$\Delta f_{pA} = \frac{2\Delta f_{pF} X}{L}$$

where:

$$X = \sqrt{\frac{E_p(\Delta L)L}{12\Delta f_{pF}}}$$

where:

X = anchor set length, ft
L = span length, ft
ΔL = 3/8 inch (anchor set)

5.9.3.3 Approximate Estimate of Time-Dependent Losses

Approximate estimate of time-dependent losses described in the latest edition of AASHTO LRFD Article 5.9.3.3 shall be used to calculate long term loss of prestress forces due to shrinkage and creep of concrete, and relaxation of prestressing steel for all precast pretensioned conventional bridge design.

For girders other than those made with composite slabs, the time-dependent prestress losses resulting from creep and shrinkage of concrete and relaxation of steel shall be determined using the refined method.

For post-tensioning with 145 to 160 ksi high strength bars, loss due to relaxation should be based on approved test data. If test data are not available, the loss may be assumed to be 3.0 ksi.

5.9.3.4 Refined Estimates for Time-Dependent Losses

For precast pretensioned girders without a composite topping and for precast or cast-in-place non-segmental post-tensioned girders the provisions of Articles 5.9.3.4.4 and 5.9.3.4.5 shall be considered.

5.9.3.4.3 Losses: Time of Deck Placement to Final Time

5.9.3.4.3b Creep of Girder Concrete

For creep of concrete, the change in prestress due to creep of girder concrete between time of deck placement and final time, Δfp·C·D, should be calculated using the total dead load applied after prestressing, including the 25 psf future wearing surface.

5.9.4 Details of Pretensioning

5.9.4.1 Minimum Spacing of Pretensioning Strand

Center-to-center spacing of strands shall be 2 inches. The use of bundled pretensioning strands shall not be allowed.

5.9.5 Detail for Post-Tensioning

5.9.5.1 Minimum Spacing of Post-Tensioning Tendons and Ducts

The clear distance between straight post-tensioning ducts in the vertical direction shall not be less than 1 inch and the clear distance in the horizontal direction shall follow the AASHTO LRFD specifications.

The use of bundled ducts shall not be allowed. In post-tensioned box girder bridges, ducts shall be arranged in vertical alignments only.

5.9.5.6 Post-Tensioned Anchorage Zones

5.9.5.6.6 Special Anchorage Devices

ADOT Bridge Group will waive the testing requirements for the special anchorage devices provided that these anchorage systems have been tested and approved for use by California Department of Transportation (Caltrans). All documentation including the test results and the acceptance certificate from Caltrans must be provided by the vendor as part of Post-Tensioned shop drawing submittal for the special anchorage devices. Reinforcements provided must be the same as was used by Caltrans during testing.

Design engineers are encouraged to communicate with the Post-Tension supplier in the design phase to obtain recommendations for the minimum cover, spacing, and edge distances for an anchorage device that may be used. This will facilitate the design of the local and the general zones. Design engineers must show dimensions of general anchor zones in the contract plans. The local anchor zone shall be displayed as a schematic in the contract plans. Post-Tension supplier must show the local anchorage zone reinforcement to be supplied as a part of any proprietary Post-Tension system (special anchorage device). Any adjustments to the general anchorage zone tensile reinforcement due to change in the local zone dimensions shall be considered as part of the shop drawing approval process.

Anchorage Zones

A 4″x 4″ grid of #4 reinforcing behind the anchorage plate shall be used and detailed on the plans. When an anchorage device requires spiral, supplemental, or both, these approved reinforcements shall be in addition to the #4 grid. When a spiral on the end anchorage of a tendon conflicts with the grid system, the reinforcements in the grid may be re-spaced or cut as required.

C-shaped reinforcing consisting of #6 at 4" spacing with 3'-0 tails shall be placed along the exterior face of the exterior web for the length of the diaphragm to aid in resisting bursting stresses.

5.10 Reinforcement

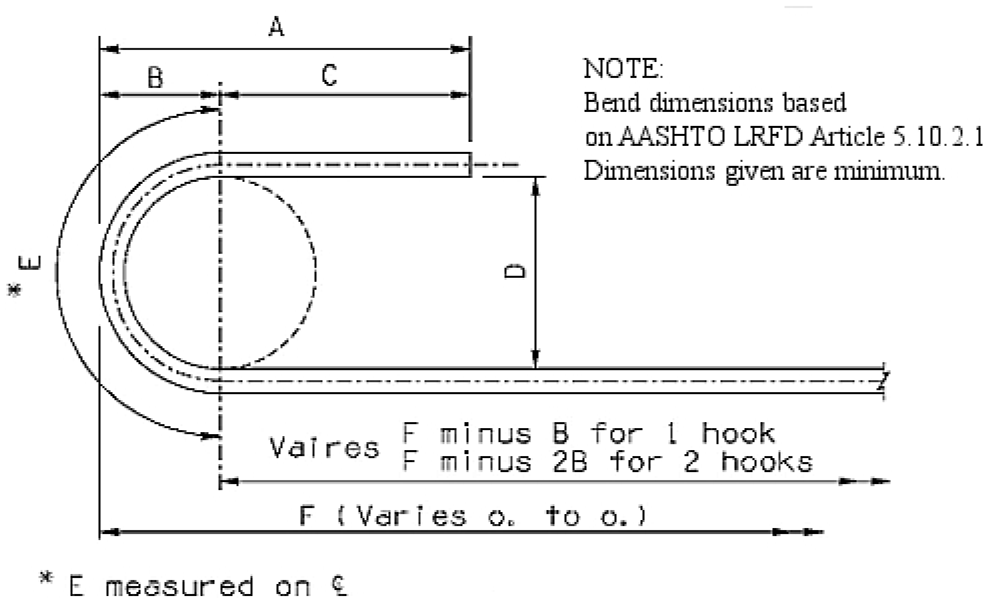
Please note that AASHTO LRFD specifications refer to main reinforcing as longitudinal reinforcing. All other reinforcing such as spirals, ties and stirrups are referred to as transverse reinforcing.

5.10.2 Hooks and Bends

5.10.2.1 Standard Hooks

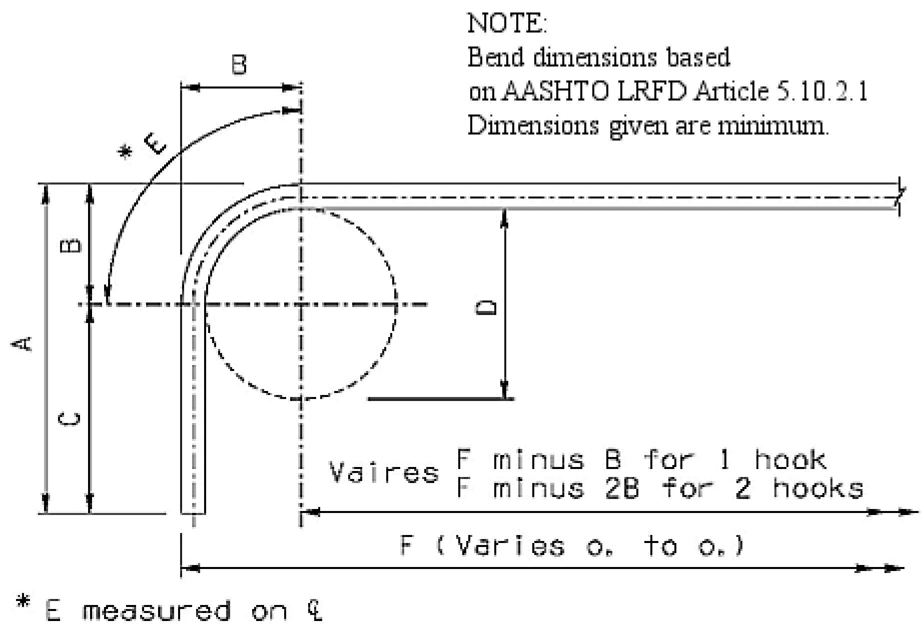
The following figures and tables shall be used:

180° Standard Hook Dimensions for Longitudinal Reinforcement



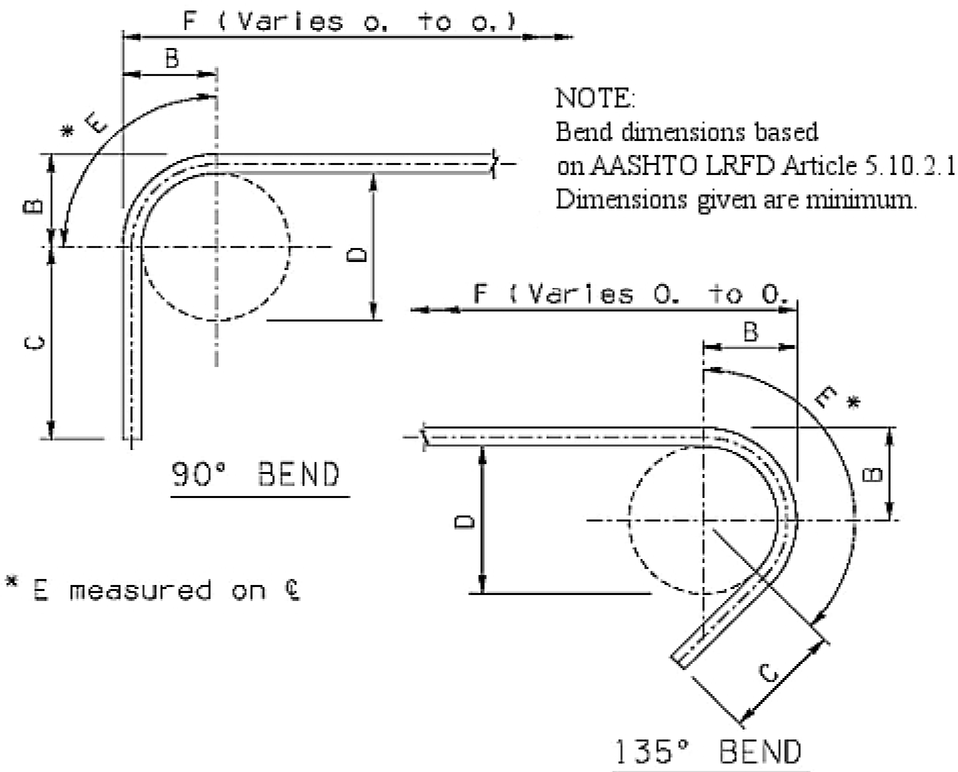
BEND DIMENSIONS (inches)								
BAR No.	BAR Dia.	D	C	B	A	E	TOTAL BAR LENGTH ADD TO F	
							1 HOOK	2 HOOKS
#3	0.375	2 ¼	4 1/2	1 ½	4	4 1/8	5 1/8	10 ¼
#4	0.500	3	2 1/2	2	4 1/2	5 1/2	6	12
#5	0.625	3 ¾	2 1/2	2 1/2	5	6 7/8	6 7/8	13 ¾
#6	0.750	4 1/2	3	3	6	8 ¼	8 ¼	16 ½
#7	0.875	5 ¼	3 1/2	3 1/2	7	9 5/8	9 5/8	19 ¼
#8	1.000	6	4	4	8	11	11	22
#9	1.128	9	4 1/2	5 5/8	10 1/8	15 7/8	14 ¾	29 ½
#10	1.270	10 1/8	5 1/8	6 3/8	11 ½	17 7/8	16 5/8	33 ¼
#11	1.410	11 ¼	5 5/8	7	12 5/8	19 7/8	18 ½	37
#14	1.693	17	6 ¾	10 ¼	17	29 3/8	25 7/8	51 ¾
#18	2.257	22 5/8	9	13 5/8	22 5/8	39 1/8	34 ½	69

90° Standard Hook Dimensions for Longitudinal Reinforcement



BEND DIMENSIONS (inches)								
BAR No.	BAR Dia.	D	C	B	A	E	TOTAL BAR LENGTH ADD TO F	
							1 HOOK	2 HOOKS
#3	0.375	2 ¼	4 1/2	1 ½	6	2	5	10
#4	0.500	3	6	2	8	2 ¾	6 ¾	13 ½
#5	0.625	3 ¾	7 1/2	2 1/2	10	3 1/2	8 1/2	17
#6	0.750	4 1/2	9	3	12	4 1/8	10 1/8	20 ¼
#7	0.875	5 ¼	10 1/2	3 1/2	14	4 3/4	11 3/4	23 1/2
#8	1.000	6	12	4	16	5 1/2	13 1/2	27
#9	1.128	9	13 1/2	5 5/8	19 1/8	8	15 7/8	31 ¾
#10	1.270	10 1/8	15 1/4	6 3/8	21 5/8	9	17 7/8	35 ¾
#11	1.410	11 ¼	16 7/8	7	23 7/8	10	19 7/8	39 ¾
#14	1.693	16 7/8	20 3/8	10 1/8	30 1/2	14 5/8	24 7/8	49 ¾
#18	2.257	22 5/8	27 1/8	13 5/8	40 3/4	19 1/2	33	66

Standard Hook Dimensions for Transverse Reinforcement



BEND DIMENSIONS (inches)											
BAR No.	BAR Dia.	D	C 90°	C 135°	B	E 90°	E 135°	TOTAL BAR LENGTH ADD TO F			
								1-90°	2-90°	1-135°	2-135°
#3	0.375	1 ½	2 ¼	2 ¼	1 1/8	1 1/2	2 1/4	2 5/8	5 1/4	3 3/8	6 3/4
#4	0.500	2	3	3	1 1/2	2	3	3 1/2	7	4 1/2	9
#5	0.625	2 ½	3 3/4	3 3/4	1 7/8	2 1/2	3 5/8	4 3/8	8 3/4	5 1/2	11
#6	0.750	4 1/2	9	4 1/2	3	4 1/8	6 1/4	10 1/8	20 ¼	7 3/4	15 1/2
#7	0.875	5 ¼	10 1/2	5 1/4	3 1/2	4 3/4	7 1/4	11 3/4	23 1/2	9	18
#8	1.000	6	12	6	4	5 1/2	8 1/4	13 1/2	27	10 1/4	20 1/2

In addition to the above tables, development of standard hooks in tension shall follow AASHTO LRFD Article 5.10.8.2.4.

5.10.3 Spacing of Reinforcement

5.10.4 Transverse Reinforcement for Compression Members

5.10.4.2 Spirals

Welded wire reinforcement shall not be used as spirals. Welded splices shall meet the welding criteria specified in AASHTO / AWS D1.5 current edition and ADOT Standard Specifications for Road and Bridge Construction.

5.10.8 Development And Splices Of Reinforcement

5.10.8.2 Development of Reinforcement

5.10.8.2.1 Deformed Bars and Deformed Wire in Tension

5.10.8.2.1a Tension Development Length

The table below, in addition to subsection 5.10.8.2.1b of these guidelines shall be used for determining development length for deformed bars.

The values of basic development length for deformed bars in tension (inch) listed in the following table may be used in lieu of the equations specified in AASHTO LRFD Article 5.10.8.2.1a:

Bar Size (f_y = 60 ksi)	**f'_c=3.0 ksi**	**f'_c=3.5 ksi**	**f'_c=4.5 ksi**	**f'_c=5.0 ksi**	**f'_c=6.0 ksi**
#3	12.0*	12.0*	12.0*	12.0*	12.0*
#4	12.0	12.0	12.0	12.0	12.0
#5	15.0	15.0	15.0	15.0	15.0
#6	19.3	18.0	18.0	18.0	18.0
#7	26.3	24.3	21.5	21.0	21.0
#8	34.6	32.0	28.3	26.8	24.5
#9	43.8	40.6	35.8	33.9	31.0
#10	55.6	51.5	45.4	43.1	39.3
#11	68.4	63.3	55.8	52.9	48.3
#14	93.6	86.6	76.4	72.5	66.2
#18	121.3	112.3	99.0	94.0	85.8

* The calculated development length, ld, for #3 bar is 9”; however, l_d shall not be less than 12” except for computation of lap splices in tension in AASHTO LRFD Article 5.10.8.4.3a, and in calculation of closed stirrup requirements in AASHTO LRFD Article 5.10.8.2.6d.

5.10.8.2.1b Modification Factors That Increase l_d

The basic development length l_d shall be multiplied by the following factors, as applicable:

Modification Factors	
Top horizontal reinforcement so placed that more than 12 inches of fresh concrete is cast below the reinforcement.	1.4
Epoxy coated reinforcements with cover: ▪ Less than 3 reinforcing bar diameter (d_b) cover or 6 d_b clear spacing between reinforcements ▪ All other cases	 1.5* 1.2
Lateral spacing $\geq$ 6” with minimum 3” clear cover in the direction of the spacing	0.8
Excess reinforcement: (As required) $\leq$ (As provided)	$(As_{req})/(As_{prov})$
Enclosed within spirals (≥ 1/4” Φ and ≤ 4” pitch)	0.75

* The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy coated reinforcement need not be taken to be greater than 1.7.

5.12 Provisions For Structure Components And Types

5.12.2 Slab Superstructures

5.12.2.1 Cast-in-Place Solid Slab Superstructures

For service limit state, the allowable tensile stress in reinforcing steel, fs, shall be limited to 28 ksi.

5.12.2.2 Cast-in-Place Voided Slab Superstructures

For service limit state, the allowable tensile stress in reinforcing steel, fs, shall be limited to 28 ksi.

5.12.2.3 Precast Deck Bridges

Prestressed Precast Box Beams

End Blocks:

End blocks 18 inches long shall be provided at each end and sufficient mild reinforcing shall be provided in the end blocks to resist the tensile forces due to the prestressing loads.

Diaphragms:

Diaphragms, cast within the beam, shall be provided at the midspan for spans up to 50 feet, at the third points for spans from 50 to 75 feet and at quarter points for spans over 75 feet.

One lateral tie shall be provided through each diaphragm located at the mid-depth of the section. However, for 39-inch or deeper sections, when adjacent units are tied in pairs for skewed bridges, in lieu of continuous ties, two ties shall be provided, located at the third points of the section depth.

Shear Keys:

After shear keys have been filled with an approved non-shrink grout and reached a minimum strength of 5.0 ksi , lateral ties shall be placed and tightened.

Prestressed Precast Voided Slabs

End Block:

End blocks should be 15 inches long with sufficient mild reinforcing provided to resist the tensile forces due to concentrated prestressing loads.
Diaphragms:

Diaphragms shall be cast within the slab at midspan for spans up to 40 feet and at third points for spans over 40 feet.

One lateral tie shall be provided through each diaphragm located at the mid-depth of the section.

The Engineer shall verify potential conflicts with lateral tie sleeves and prestressing strands when harped strands are used.

5.12.2.3.3 Shear-Flexure Transfer Joints

Lateral Ties:

Each tie shall consist of a 1½-inch diameter mild steel bar tensioned to 30,000 pounds. Tension in the 1½-inch diameter mild steel should be applied by the turn of nut method. The design engineer should determine the number of turns of the nut required to achieve the 30,000 pounds force. This value should be shown on the plans.

ASTM A36 steel bars for the tie normally come in 20-foot lengths. The final total length of the tie should be made using threaded couplers; not welded splices. When couplers are used, the hole through the diaphragm should be increased from the normal 2½ inches to 4 inches diameter to accommodate the couplers.

Adequate means shall be used to ensure that the ties are adequately protected from corrosion. The rod, nut and bearing plate shall be galvanized in accordance with ASTM A153 (AASHTO M-232).

5.12.2.3.3f Structural Overlay

For precast deck bridges, a structural concrete overlay shall be applied. The thickness of structural concrete overlays shall be 5 inches minimum, and shall be designed for strength and service analysis to exclude a ½" wearing surface.

5.12.3 Beams and Girders

5.12.3.2 Precast Beams

Precast prestressed girders shall be designed as simply supported beams using composite section properties for dead and transient loads. Superstructure constructed as continuous over intermediate supports shall be designed for transient and composite dead load. The design should include the effects of shrinkage and creep for all strength limit states. Additional non-prestressed reinforcement shall be provided in the deck slab to account for continuity over the intermediate supports. The design shall be based on the strength of concrete of the closure joint. Additional continuity reinforcement shall be designed per Section 5, AASHTO LRFD Bridge Design Specifications including Article 5.10.8.1.2c ,5.12.3.3.8 , and 5.12.3.3.9.

Precast prestressed girder spacing shall not exceed 12.0 feet center to center of girders unless approved by ADOT Bridge Group.

Differential shrinkage shall be considered in the design.

5.12.3.3 Bridges Composed of Simple Span Precast Girders Made Continuous

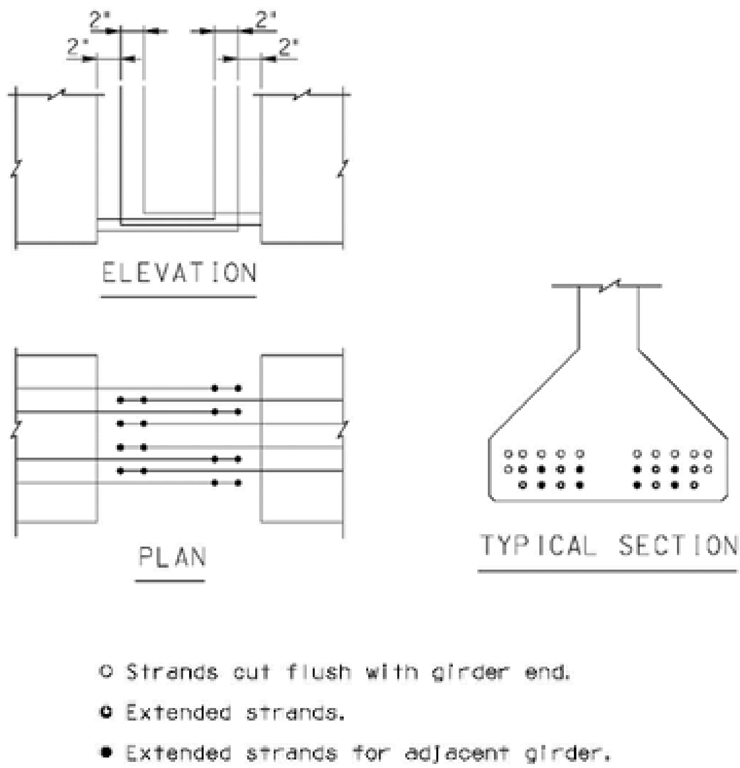
The girders shall be designed as simply supported beams for dead load and live load plus dynamic load allowance.

In-lieu of more refined analysis it is acceptable to use the following:

Design for 1.2·Mcr for determining the number of extended strands for the positive moment connection for simple span precast girders made continuous. Since it is difficult to control the duration between casting of the girders and casting of the pier diaphragm, which has such a large effect on the analysis, it is acceptable to limit the design force to 1.2·Mcr.

In determining the number and pattern of strands extended, preference shall be given to limiting the number of strands by increasing the extension length and alternating the pattern to increase constructability. The minimum clearance of an extended strand to the end of an adjacent girder shall be 2 inches.

STRAND PATTERN AT GIRDER END



Method of Analysis for Precast I-Girders

Section properties shall be based on the transformed area of bonded prestressing strands for precast prestressed members.

The girders shall be designed as simply supported beams for dead load and live load plus dynamic load allowance.

Girders shall be designed using the pretensioning method only. Post-tensioned alternates shall be used only for projects with unusual constraints when approved by ADOT Bridge Group.

Debonding shall not be allowed.

The location of the harped points of the strand should be as required by design with the preferable locations being near the 1/10 of the span as measured from the midspan of the girder.

The theoretical build-up depth shall be ignored for calculation of composite section properties, but shall be included in the DC load on the girder.

5.12.3.5 Cast-in-Place Box Girders and T-Beams

For service limit state, the allowable tensile stress in reinforcing steel, fs, shall be limited to 28 ksi.

5.12.3.5.1 Flange and Web Thickness

5.12.3.5.1a Top Flange

Minimum top flange thickness shall be 8 inches, refer to Section 9 of these guidelines.

5.12.3.5.1b Bottom Flange

Minimum bottom flange thickness shall not be less than 6 inches.

5.12.3.5.1c Web

Web thickness shall not be less than 12 inches (measured normal to girder for sloping exterior webs). Exterior girders webs shall be flared to a minimum thickness of 18 inches at the abutments. The flare length shall be 16 feet from the face of the abutment diaphragm. Interior webs shall be constructed vertical. A 4″x 4″ fillet shall be used at the tops of webs but is not required at the bases.

The minimum web thickness shall be 14 inches for girders over 10.0 feet in depth.

Method of Analysis for Cast-In-Place Box Girder

Section properties shall be based on gross area of members for cast-in-place post-tensioned members.

The bottom slab, in the vicinity of the intermediate support, may be flared to increase its thickness at the face of the support when the required concrete strength exceeds 4.5 ksi. When thickened, the bottom slab thickness should be increased by a minimum of 50 percent. The length of the flare should be at least one-tenth of the span length (measured from the center of the support) unless design computations indicate that a longer flare is required.

Section properties at the face of the support should be used throughout the support; i.e., the solid cap properties should not be included in the model.

Negative moments should be reduced to reflect the effect of the width of the integral support.

The combination of dead load and prestress forces should not produce any tension in the extreme fibers of the superstructure.

Cast-in-Place multi-cell concrete box girder bridges shall be designed as one unit for the entire cross-section of the superstructure. Such cross-sections shall be designed for live load distribution factors specified in AASHTO LRFD Articles 4.6.2.2.2 and 4.6.2.2.3 for interior girders, multiplied by the number of girders, i.e., webs.

For box girders with severe sloping webs or boxes over 7 feet deep, transverse flange forces induced by laterally inclined longitudinal post-tensioning shall be considered in the design.

Single span structures should be jacked from one end only. Symmetrical two span structures may be jacked from one end or from both ends. Unsymmetrical bridges should be jacked from the long end only or from both ends as required by the design. Three span or longer structures should be jacked from both ends. The engineer shall state the required jacking specifications and sequence on the plans.

Several prestressing systems should be checked to verify that the eccentricity and anchorage details are acceptable. In determining the center of gravity of the strands, the difference between the center of gravity of the strands and the center of the ducts, shall be considered. For structures over 400 feet in length, in determining the center of gravity of the strands, the diameter of the ducts should be oversized by ½ inch to allow for ease of pulling the strands.

For horizontally curved bridges, special care shall be taken in detailing stirrups and duct ties. Loss of prestress due to friction should be based on both vertical and horizontal curvatures. In designing for horizontal curvature, the exterior web with the smallest radius shall be used. A variation of prestressed force not to exceed 5% per web shall be allowed provided that total jacking force remains the same as calculated in design.

For concrete bridge widenings implementing Post-Tensioned Cast-In-Place girder bridges, there shall be notes and details in the contract plans requiring a closure pour in the deck between the new and existing superstructure, with a minimum wait time of 60 days after final post-tensioning.

5.12.4 Diaphragms

For post-tensioned box girder bridges, a single 9-inch thick intermediate diaphragm shall be placed at the midspan. Special consideration for additional diaphragms shall be given to box girders with large skews, curved boxes and boxes over seven feet in depth. Diaphragms shall be placed parallel to abutments and piers for skews less than or equal to 20 degrees. Diaphragms shall be placed normal to girders and staggered for skews over 20 degrees. All diaphragms shall be cast integral with girder webs to add lateral stability to the forming system.

For prestressed precast I-Girder bridges, a single 9-inch thick intermediate diaphragm shall be placed at the midspan for all spans over 40 feet. For skews less than or equal to 20 degrees, the diaphragm shall be placed parallel to the skew. For skews greater than 20 degrees, the diaphragms shall be staggered and placed normal to the girder.

The use of intermediate steel diaphragms for precast I-Girder bridges are only allowed for use on bridges over a railroad. Steel diaphragms shall be K-Frame and detailed to fit up with embedded plates cast into the precast girder.

5.13 Anchors

5.13.1 General

Designers shall only use adhesive post-installed anchors for connecting new elements to existing concrete.

Designers shall not use adhesive anchorage systems which are constantly under significant sustained tensile loads in vertical overhead applications. Features such as overhead lighting systems and conduit support applications for small utilities (excluding heavier elements such as water and sewer), are acceptable given the provisions in AASHTO 5.13 are followed.

Designers shall only use cast-in-place anchors for new concrete construction, unless otherwise approved by Bridge Group.