**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 bars, welded wire reinforcement, prestressing strands, prestressing bars, or prestressing wires. Concrete deck design criteria are specified in Section 9 of these guidelines.

All design engineers are advised to review the example problems in Appendix – A of these guidelines for proper and correct application of various provisions of the AASHTO LRFD Specifications and these guidelines for design of bridge structural components.

Minimum vertical clearance for a bridge should be established based on future roadway configuration. For bridges spanning over railroads, minimum vertical clearance shall be based on the most recent railroad grade separation guidelines.

Design level load ratings of all bridges shall be performed per MBE (The Manual for Bridge Evaluation) 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, unless approved by ADOT Bridge Group for bridges with specific circumstances. For widening of bridges the minimum design Operating Load Rating (using HL93 live load) shall be the Operating Load Rating (using HL93 live load) of the existing bridge or 1.5, whichever is greater. Coordination and approval from ADOT Bridge Group will be required in instances where these provisions cannot be met, for widening of existing bridges.

## 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.

Designers shall not use epoxied anchorage systems which are constantly under pure axial tension.

### 5.4.2 Normal and Structural 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. Normal weight non-prestressed concrete shall have minimum strengths, $f'_c$, at 28 days as follows:

| Components | $f'_c$ (ksi) |
|---|---|
| Decks except barriers | 4.5 |
| Bridge concrete barriers | 4.0 |
| Substructures (abutments, piers, foundations and drilled shafts) | 3.5 |
| All other Class 'S' Concrete | 3.0 |

Prestressed Precast Concrete shall have 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* | $f'_{ci}$ = 5.0 ksi | $f'_c$ = 6.5 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.

5.4.2.3 Shrinkage and Creep

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

Precast Prestressed Application:
Approximate Estimate of Time-Dependent Losses described in the current edition of AASHTO LRFD Article 5.9.5.3, shall be used for all precast pretensioned conventional applications (spans not to exceed 140'-0"). For spans over 140'-0", ADOT Bridge Group shall be consulted regarding the loss calculation method to be used.

Post-Tensioned Application:
In lieu of a more detailed analysis, prestress losses in post-tensioned members, constructed and prestressed in a single stage may be based on AASHTO LRFD Article 5.9.5, third edition, 2004, the details of which are reproduced in Article 5.9.5 of these guidelines and are demonstrated in the examples in Appendix A for convenience.

5.4.2.4 Modulus of Elasticity

Based on an assumed unit weight of concrete of $w_c = 0.145$ kcf, the modulus of elasticity, $E_c$, 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 $w_c = 0.150$ kcf.

5.4.3 Reinforcing Steel

All reinforcing steel shall be supplied as Grade 60 and shall be deformed bars conforming to ASTM A 615 / A 615M-96a, except for smooth wire spiral ties. Spiral ties shall be cold drawn wire conforming to ASTM A 82 (AASHTO M-32).

Welded wire fabric reinforcing shall only be used in slope paving and prefabricated panels used for sound walls. Welded wire reinforcing shall be deformed. Deformed welded steel wire and fabric shall conform to ASTM A 496 and ASTM A 497.

For structural applications in places other than slope paving or prefabricated panels, only deformed steel bars conforming to ASTM A 706 shall be used if approved by ADOT Bridge Group. Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A 706 shall be specified.

All new bridge construction located above an elevation of 4,000 feet, or for areas where de-icing chemicals are used, deck slabs, barriers, anchor slabs and approach slabs reinforcing as well as portions of reinforcement projecting into the deck slabs shall be epoxy coated. Reinforcing bars shall conform to the requirements of ASTM A 775 or ASTM A 934. Welded wire fabric reinforcing shall conform to ASTM A 884, Class A.

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 diameter strands should 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

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 $\phi = 0.95$. AASHTO LRFD Article 5.5.4.2 shall be used for all other cases.

## 5.7 Design For Flexural and Axial Force Effects

5.7.2 Assumptions for Strength and Extreme Event Limit States

5.7.2.1 General

Unbonded tendons shall not be used.

5.7.3 Flexural Members

5.7.3.2 Flexural Resistance

For post-tensioned box girders, in determining the flexural resistance, 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.

5.7.3.4 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, $f_s$, shall be limited to 30 ksi, 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.

5.7.3.6 Deformations

5.7.3.6.1 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 |

The design engineer may determine the deflections due to creep and shrinkage based on the next article of these guidelines (5.7.3.6.2). Creep and shrinkage values presented in AASHTO LRFD Article 5.4.2.3 may also be used for more precise determination of long-term deflections as suggested in AASHTO LRFD Commentary 5.7.3.6.1. If the method specified in AASHTO LRFD Article 5.4.2.3 is used, the calculation should include construction stage analysis considering transition of section properties, maturity of concrete, prestress losses, and change of structural model, that is, transition from simple to continuous span.

5.7.3.6.2 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 I-girder:

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. Design engineers are advised to review Appendix – A, Example problems No. 3 and 4 for deflection calculation details.

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, as specified in AASHTO LRFD Article 5.9.5.3, third edition, 2004. 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 Type III and smaller girders shall be ½ inch. For Type IV, V and VI girders the minimum build-up 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 prior to the erection of the girders.

## 5.8 Shear and Torsion

### 5.8.2 General Requirements

#### 5.8.2.6 Types of Transverse Reinforcement

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

### 5.8.3 Sectional Design Model

Note that $N_u$ in AASHTO LRFD Articles 5.8.3.4.2 and 5.8.3.5 is not the prestress force but is the externally applied factored axial force, taken positive if tensile.

5.8.3.5 Longitudinal Reinforcement

AASHTO LRFD Equation 5.8.3.5-1 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 5.8.3.5-2 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 5.8.3.2). The values of $V_u$, $V_s$, $V_p$, and $\theta$, calculated for the section $d_v$ from the face of the support, may be used.

5.8.4 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 I-Girders, the following note shall be shown on the plans: The contact surface of the top flange 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 and Partial Prestressing

5.9.1 General Design Considerations

5.9.1.4 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 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.

5.9.3 Stress Limitations for Prestressing Tendons

For post-tensioned members, overstressing for 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 $0.78f_{pu}$.

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

5.9.4 Stress Limits for Concrete

| | | Load Cases | | | | |
|---|---|---|---|---|---|---|
| | | Before Time-Dependent Losses | After Losses | | | |
| | | | DC + Prestress | Service Limit I | Service Limit III | 0.5(DW+DC + Prestress) + (LL + IM) |
| **Compression (ksi)** | | $0.6 f_{ci}^{'}$ | $0.45 f_{c}^{'}$ | $0.6\phi_w f_{c}^{'}$ | N/A | $0.4 f_{c}^{'}$ |
| **Tension (ksi)** | Any region of a prestressed component in which prestressing causes compressive stresses and service load effects cause tensile stresses | N/A | 0 for post-tensioned boxes<br><br>N/A for precast prestressed members | N/A | $0.0948\sqrt{f_{c}^{'}}$ (For post-tensioned structures built on falsework, this value shall be zero. No tension shall be allowed) | N/A |
| | Other Regions | $0.0948\sqrt{f_{ci}^{'}} \leq 0.2ksi$ | N/A | N/A | N/A | N/A |

5.9.5 Loss of Prestress

In lieu of a more detailed analysis for determining prestress losses in post-tensioned members constructed and prestressed in a single stage, relative to the stress immediately before transfer, the AASHTO LRFD Specifications, third edition, 2004 method may be used. The method is reiterated below for convenience.

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

where:

$\Delta f_{pT}$ = total loss (ksi)

$\Delta f_{pF}$ = loss due to friction (ksi)

$\Delta f_{pA}$ = loss due to anchorage set (ksi)

$\Delta f_{pES}$ = loss due to elastic shortening (ksi)

$\Delta f_{pSR}$ = loss due to shrinkage (ksi)

$\Delta f_{pCR}$ = loss due to creep of concrete (ksi)

$\Delta f_{pR2}$ = loss due to relaxation of steel after transfer (ksi)

Also, consideration should be given to a loss of tendon force, as indicated by pressure readings, within the stressing equipment.

Loss Due to 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 the “Friction Coefficients for Post-Tensioning Tendons” table below.

Losses due to friction between the internal prestressing tendons and the duct will be taken as:

$$\Delta f_{pF} = f_{pj}\,(1 - e^{-(Kx + \mu\alpha)}) \qquad (5.9.5.2.2b\text{-}1)$$

where:

$f_{pj}$ = stress in the prestressing steel at jacking (ksi)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft)
K = 0.0002, wobble friction coefficient (per ft of tendon)
μ = coefficient of friction
α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad)
e = base of Napierian logarithms

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 below. In the absence of such data, a value of μ = 0.25 may be used.

For tendons confined to a vertical plane, α shall be taken as the sum of the absolute values of angular changes over length x.

For tendons curved in three dimensions, the total three-dimensional angular change α shall be obtained by vectorially adding the total vertical angular change, $\alpha_v$ and the total horizontal angular change, $\alpha_h$.

Friction Coefficients for Post-Tensioning Tendons

| Type of Steel | Type of Duct | K | μ |
|---|---|---|---|
| **Wire or Strand** | Rigid and semi-rigid Galvanized metal sheathing | 0.0002 | 0.15-0.25 |
| | Polyethylene | 0.0002 | 0.23 |
| **High-strength bars** | Galvanized metal sheathing | 0.0002 | 0.30 |

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)

Loss Due to Elastic Shortening

In calculating the loss due to elastic shortening, $\Delta f_{pES}$, in post-tensioned members other than slab systems, the following corrected equation shall be used:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + A_g e_m^2\right) - e_m M_g A_g}{A_{ps}\left(I_g + A_g e_m^2\right) + \frac{A_g \cdot I_g \cdot E_{ci}}{E_p}\left(\frac{2N}{(N-1)}\right)} \qquad \text{(C5.9.5.2.3b-1)}$$

where:

$A_{ps}$ = area of prestressing steel (in$^2$)
$A_g$ = gross area of section (in$^2$)
$E_{ci}$ = modulus of elasticity of concrete at transfer (ksi)
$E_p$ = modulus of elasticity of prestressing steel (ksi)
$e_m$ = average eccentricity at mid span (in)

$f_{pbt}$ = stress in prestressing steel immediately prior to transfer as specified in AASHTO LRFD Table 5.9.3-1, (ksi)
$I_g$ = moment of inertia of gross concrete section ($in^4$)
$M_g$ = midspan moment due to member self weight (kip-in)
N = number of identical prestressing tendons

The value of $\Delta f_{pES}$ should be determined at the center section of the span for simple spans and at critical sections for continuous spans.

For slab systems, the value of $\Delta f_{pES}$ may be taken as 25 percent of that obtained from Equation C5.9.5.2.3b-1, as shown above.

For post-tensioned construction, the value of $\Delta f_{pES}$ may be further reduced below those implied by Equation C5.9.5.2.3b-1, as shown above, with proper tensioning procedures such as stage stressing and retensioning.

Loss Due to Shrinkage

Loss of prestress for post-tensioned members, in ksi, due to shrinkage may be taken as:

$$\Delta f_{pSR} = (13.5 - 0.123H)$$

where:

H = the average annual ambient relative humidity (percent), may be taken as 40%.

Loss Due to Creep of Concrete

Prestress loss, in ksi, due to creep may be taken as:

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0\,\Delta f_{cdp} \geq 0$$

where:

$f_{cgp}$ = concrete stress at center of gravity of prestressing steel at transfer (ksi)
$\Delta f_{cdp}$ = change in concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied.

Values of $\Delta f_{cdp}$ should be calculated at the same section or at sections for which $f_{cgp}$ is calculated.

Loss Due to Relaxation of Steel after Transfer

For post-tensioning with stress-relieved strands:

$$\Delta f_{pR2} = 20.0 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right)$$

where:

$\Delta f_{pF}$ = the friction loss below the level of 0.70 $f_{pu}$ at the point under consideration as computed above (ksi)

$\Delta f_{pES}$ = loss due to elastic shortening (ksi)

$\Delta f_{pSR}$ = loss due to shrinkage (ksi)

$\Delta f_{pCR}$ = loss due to creep of concrete (ksi)

For prestressing steels with low relaxation properties conforming to ASTM A 416 or E 328 (AASHTO M 203) use 30 percent of $\Delta f_{pR2}$ .

For post-tensioning with 145 to 160 ksi 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.

For post-tensioned members with:

Spans not greater than 250 ft,
Normal weight concrete, and
Strength in excess of 3.50 ksi at the time of prestress,

Values of creep, shrinkage, and relaxation related losses, may be determined in accordance with the provision of AASHTO LRFD Article 5.4.2.3 or this article.

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.5 of these guidelines, including consideration of the time-dependent construction method and schedule shown in the contract documents.

For precast pre-tensioned members other than slab systems, in calculating the loss of prestress due to elastic shortening, $\Delta f_{pES}$, design engineers shall use AASHTO LRFD equation C5.9.5.2.3a-1 of Article 5.9.2.3.

5.9.5.3 Approximate Estimate of Time-Dependent Losses

Approximate estimate of time-dependent losses described in the latest edition of AASHTO LRFD Article 5.9.5.3 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 structures with spans greater than 140 feet, ADOT Bridge Group shall be consulted for the appropriate method of time-dependent loss calculation to be used.

The average annual ambient relative humidity, H, shall be 40% for calculation of concrete creep and shrinkage coefficients whenever applying AASHTO LRFD Article 5.4.2.3. H shall also be 40% whenever applying both approximate and refined estimates of prestress loss calculations in AASHTO LRFD Articles 5.9.5.3 and 5.9.5.4 respectively.

Relaxation before Transfer

For precast prestressed concrete girders, the release time, which is the estimated time from stressing to transfer, may be assumed to be 36 hours. For precast prestressed girders, the initial and final losses shall include the release losses $\Delta f_{pR1}$. AASHTO LRFD Equation (5.9.5.1-1) should be modified as follows:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} + \Delta f_{pR1}$$

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

where:

$\Delta f_{pR1}$ = relaxation loss of prestress before transfer (ksi)

$t$ = time estimated in days from stressing to transfer

$f_{pj}$ = initial stress in the tendon at the end of stressing (ksi)

$f_{py}$ = specified yield strength of prestressing steel (ksi)

5.9.5.4 Refined Estimates for Time-Dependent Losses

Creep

For creep of concrete, the variable, $f_{cgp}$ defined in AASHTO LRFD Article 5.9.5.4.3b, should be calculated using the total dead load applied after prestressing, including the 25 psf future wearing surface.

5.10 Details of Reinforcement

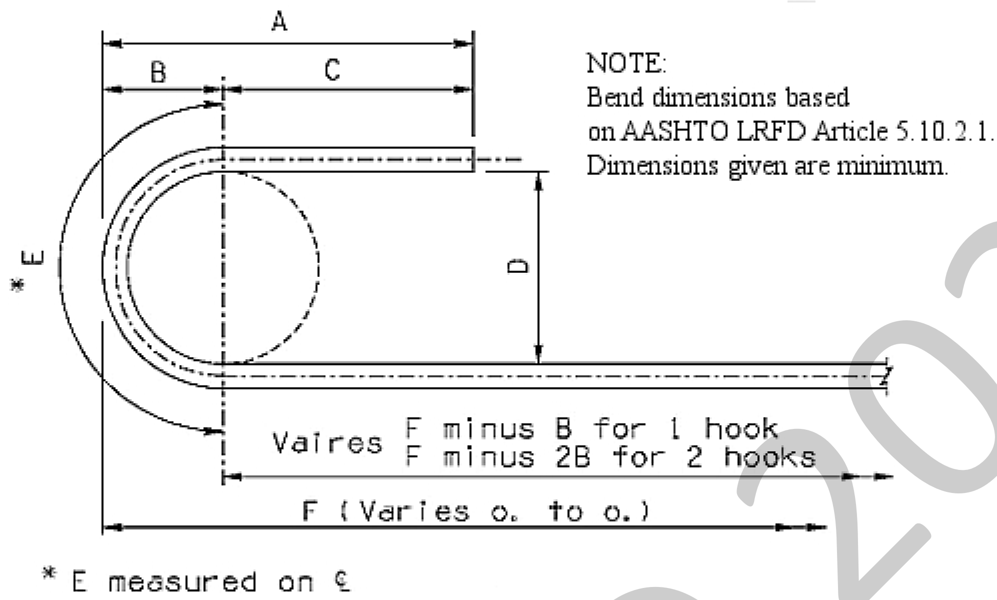
Please note that AASHTO LRFD specifications refer to main reinforcing as longitudinal reinforcing. All other reinforcing such as spirals, ties and stirrups are referred to as transverse reinforcing.

5.10.2 Hooks and Bends

5.10.2.1 Standard Hooks

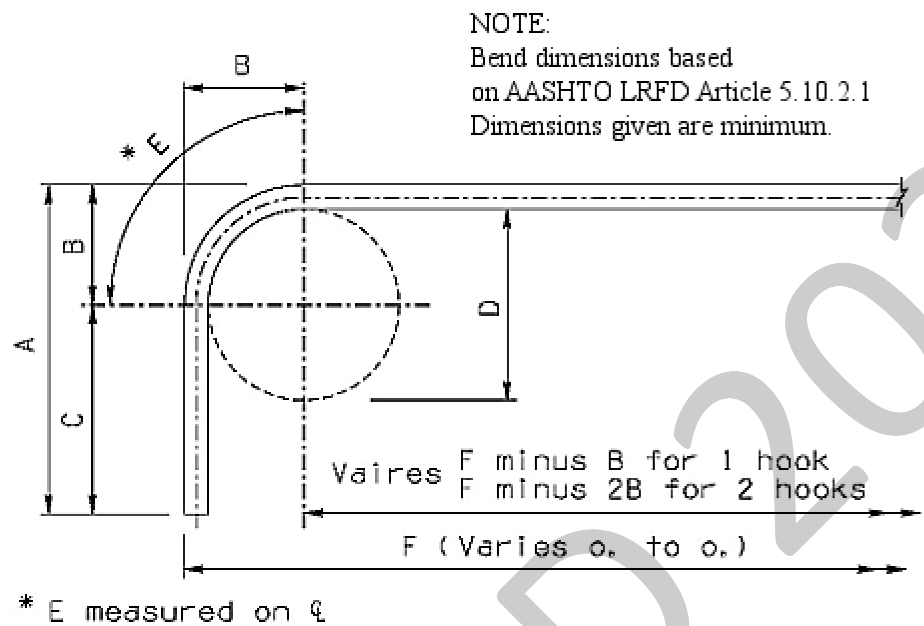
The following tables shall be used.

180º STD. Hook Dimensions for Longitudinal Reinforcement



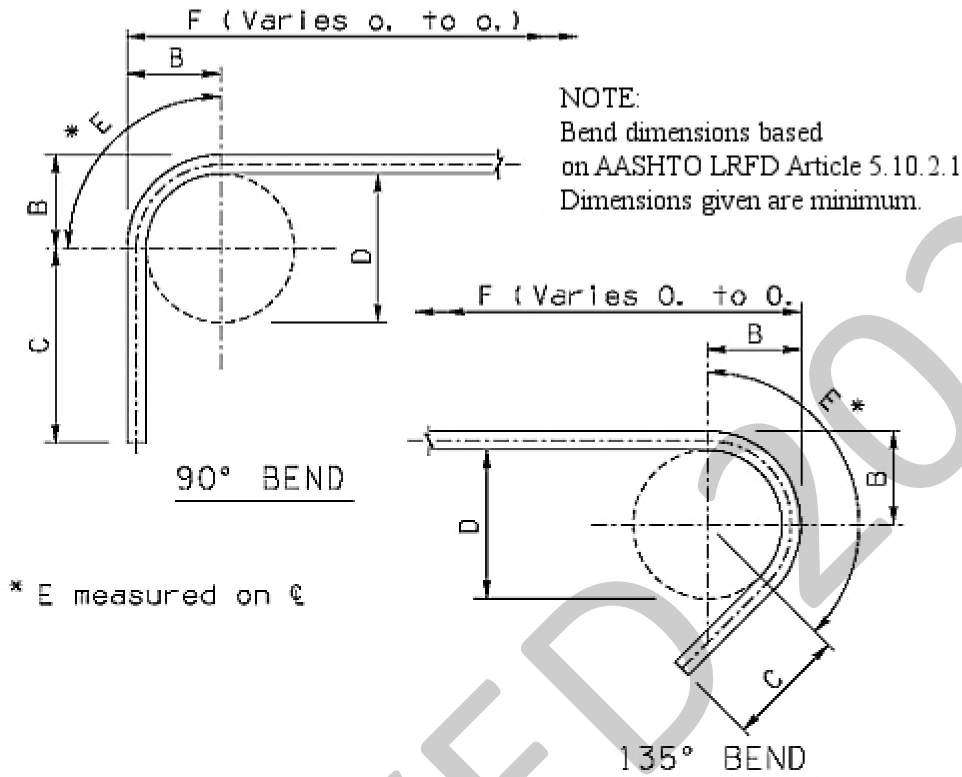

| BEND DIMENSIONS (inches) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| BAR No. | BAR Dia. | D | C | B | A | E | | | TOTAL BAR LENGTH ADD TO F 1 HOOK | 2 HOOKS |
| #3 | .375 | 2 ¼ | 2½ | 1 ½ | 4 | 4 1/8 | | | 5 1/8 | 10 ¼ |
| #4 | .500 | 3 | 2½ | 2 | 4 ½ | 5 ½ | | | 6 | 12 |
| #5 | .625 | 3 ¾ | 2½ | 2 ½ | 5 | 6 7/8 | | | 6 7/8 | 13 ¾ |
| #6 | .750 | 4 ½ | 3 | 3 | 6 | 8 ¼ | | | 8 ¼ | 16 ½ |
| #7 | .875 | 5 ¼ | 3½ | 3 ½ | 7 | 9 5/8 | | | 9 5/8 | 19 ¼ |
| #8 | 1.000 | 6 | 4 | 4 | 8 | 11 | | | 11 | 22 |
| #9 | 1.128 | 9 | 4 ½ | 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 |

90º STD. 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 | .375 | 2 ¼ | 4 ½ | 1 ½ | 6 | 2 | | | 5 | 10 |
| #4 | .500 | 3 | 6 | 2 | 8 | 2 ¾ | | | 6 ¾ | 13 ½ |
| #5 | .625 | 3 ¾ | 7 ½ | 2 ½ | 10 | 3 ½ | | | 8 ½ | 17 |
| #6 | .750 | 4 ½ | 9 | 3 | 12 | 4 1/8 | | | 10 1/8 | 20 ¼ |
| #7 | .875 | 5 ¼ | 10 ½ | 3 ½ | 14 | 4 ¾ | | | 11 ¾ | 23 ½ |
| #8 | 1.000 | 6 | 12 | 4 | 16 | 5 ½ | | | 13 ½ | 27 |
| #9 | 1.128 | 9 | 13 ½ | 5 5/8 | 19 1/8 | 8 | | | 15 7/8 | 31 ¾ |
| #10 | 1.270 | 10 1/8 | 15 ¼ | 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 ½ | 14 5/8 | | | 24 7/8 | 49 ¾ |
| #18 | 2.257 | 22 5/8 | 27 1/8 | 13 5/8 | 40 ¾ | 19 ½ | | | 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 T0 F | | | |
| | | | | | | | | 1-90° | 2-90° | 1-135° | 2-135° |
| #3 | .375 | 1 ½ | 2 ¼ | 2 ¼ | 1 1/8 | 1 ½ | 2 ¼ | 2 5/8 | 5 ¼ | 3 3/8 | 6 ¾ |
| #4 | .500 | 2 | 3 | 3 | 1 ½ | 2 | 3 | 3 ½ | 7 | 4 ½ | 9 |
| #5 | .625 | 2 ½ | 3 ¾ | 3 ¾ | 1 7/8 | 2 ½ | 3 5/8 | 4 3/8 | 8 ¾ | 5 ½ | 11 |
| #6 | .750 | 4 ½ | 9 | 4 ½ | 3 | 4 1/8 | 6 ¼ | 10 1/8 | 20 ¼ | 7 ¾ | 15 ½ |
| #7 | .875 | 5 ¼ | 10 ½ | 5 ¼ | 3 ½ | 4 ¾ | 7 ¼ | 11 ¾ | 23 ½ | 9 | 18 |
| #8 | 1.000 | 6 | 12 | 6 | 4 | 5 ½ | 8 ¼ | 13 ½ | 27 | 10 ¼ | 20 ½ |

In addition to the above tables, development of standard hooks in tension shall follow AASHTO LRFD Article 5.11.2.4.

5.10.3 Spacing of Reinforcement

5.10.3.3 Minimum Spacing of Prestressing Tendons and Ducts

5.10.3.3.1 Pretensioning Strand

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

5.10.3.3.2 Post-Tensioning Ducts Not Curved in the Horizontal Plane

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.

5.10.6 Transverse Reinforcement for Compression Members

5.10.6.2 Spirals

Welded wire fabric 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 Shrinkage and Temperature Reinforcement

The permanent prestress of 0.11ksi, specified in the AASHTO LRFD Commentary 5.10.8, should not be added to that required for the strength or service limit state evaluations. It is the minimum requirement for shrinkage and temperature crack control.

5.10.9 Post-Tensioned Anchorage Zones

5.10.9.3 Design of the General Zone

For design of the general anchorage zones, any of the following design methods, conforming to the requirements of AASHTO LRFD Article 5.10.9.3.2, may be used:

Equilibrium-based inelastic models, generally termed as “strut-and-tie models”

Refined elastic stress analyses as specified in Section 4 and AASHTO LRFD Article 5.10.9.5

Other approximate methods Specified in AASHTO LRFD Article 5.10.9.6

5.10.9.3.3 Special Anchorage Devices

Where special anchorage devices that do not satisfy the requirements of AASHTO LRFD Article 5.10.9.7.2 are to be used, those devices shall be tested for compliance with the requirements of AASHTO LRFD Article 5.10.9.3.3.

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 @ 4″ with 3′-0″ tails shall be placed along the exterior face of exterior web for the length of the diaphragm to aid in resisting bursting stresses.

## 5.11 Development and Splices OF REINFORCEMENT

5.11.2.1 Deformed Bars and Deformed Wire in Tension

5.11.2.1.1 Tension Development Length

The following table in addition to the following article of these guidelines shall be used for determining development length for deformed bars.

Basic Development Length For Deformed Bars in Tension (inch) is based on AASHTO LRFD Article 5.11.2.1.1

| 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, $l_d$, for # 3 bar is 9″, however $l_d$ shall not be less than 12" except in computation of lap splices by AASHTO LRFD Article 5.11.5.3.1 and in calculation for closed stirrup requirement in AASHTO LRFD Article 5.11.2.6.4.

5.11.2.1.2 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<br>▪ Less than 3 reinforcing bar diameter ($d_b$) cover or 6 $d_b$ clear spacing between reinforcements …………<br>▪ All other cases ……………………………………. | <br>1.5*<br>1.2 |
| Lateral spacing ≥ 6" with minimum 3" clear cover in the direction of the spacing | 0.8 |
| Excess reinforcement - ($As_{required}$) ≤ ($As_{provided}$) | $\frac{(As_{required})}{(As_{provided})}$ |
| 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 Durability

Concrete structures shall be designed to provide protection of the reinforcing and prestressing steel against corrosion throughout the life of the structure. Protective measures for durability shall satisfy the requirements specified in AASHTO LRFD Article 2.5.2.1. All new bridge construction located above an elevation of 4,000 feet, or for areas where de-icing chemicals are used, deck slabs, barriers, anchor slabs and approach slabs reinforcing as well as portions of reinforcement projecting into the deck slabs shall be epoxy coated.

Use of High Performance Concrete (HPC) could be considered as a measure of enhancing concrete durability if approved by ADOT Bridge Group.

## 5.13 SPECIFIC Members

5.13.2.2 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.

## 5.14 Provisions for Structure Types

5.14.1 Beams and Girders

5.14.1.1 General

Girders shall be placed accurately on bearings to avoid creating eccentricities capable of initiating imbalance.

Girders with shapes that exceed a height to width ratio of two shall be temporarily braced. The girder width shall be determined from the outside dimension of the bottom flange.

The contractor shall secure such girders in position on the structure with temporary lateral bracing to resist loads as specified in the AASHTO Guide Design Specifications for Bridge Temporary Works. Lateral bracing shall be designed to allow for girder temperature movements. The bracing shall be placed prior to the release of the erection equipment from each girder.

Prior to erection of any girders, the contractor shall provide a lateral bracing plan, prepared and sealed by a professional engineer registered in the State of Arizona, for the Engineer's review. Such bracing plan shall be included with the working drawings specified in Subsection 105.03, and shall include supporting calculations. A girder pre-erection meeting will be scheduled following the review and prior to erection of any girders. All parties involved in the installation shall be represented, and no girders shall be placed until the plan has been approved.

No traffic shall be allowed under each newly erected girder until the girder has been laterally braced.

Temporary bracing shall remain in place until after permanent concrete diaphragms are installed at the bents, or the girder is integrated with a permanent feature that restricts the girder's lateral movement.

5.14.1.2 Precast Beams

Precast prestressed girders shall be designed as simply supported beams using composite section properties for dead and transient loads. The superstructure shall be constructed continuous over the intermediate supports and 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 pour. Additional continuity reinforcement shall be designed per Section 5, AASHTO LRFD Bridge Design Specifications including Article 5.11.1.2.3, 5.14.1.4.8, and 5.14.1.4.9.

Due to the increase in the number of overweight permits, precast prestressed girder spacing shall not exceed 10.0 feet unless approved by ADOT Bridge Group.

5.14.1.2.4 Detail Design

Differential shrinkage shall be considered in the design.

5.14.1.4 Bridges Composed of Simple Span Precast Girders Made Continuous

The positive moment connection may be designed using the methods described in the PCA publication "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders", August 1969. In determining the positive restraint moment using the referenced PCA publication, the duration between casting of the girders and deck closure may be taken as 30 days.

The development length of the strands may be based on criteria contained in Report No. FHWA-RD-77-14, "End Connections of Pretensioned I-Beam Bridges" November 1974. The equations from the report are reproduced below for convenience.

$$L_e = 0.228 f_{ps} + 8.25, \qquad \text{where } L_{pb} \le 8.25"$$

$$L_e = 0.228\left[f_{ps} - \frac{L_{pb} - 8.25}{0.472}\right] + L_{pb}, \qquad \text{where } L_{pb} \ge 8.25"$$

where:

$L_e$ = embedment length of strand (inch)
$L_{pb}$ = prebend length of embedded strand (inch)
$f_{ps}$ = allowable stress per strand (inch)

The above relationship may be rewritten as

$$f_{ps} = \frac{L_e - 8.25}{0.228} \le 150\text{ksi} \qquad \text{where } L_{pb} \le 8.25"$$

The required number of strands to be extended may be determined using the following equation:

$$A_{ps(req'd)} = \frac{M^{+} - A_s f_y (jd_{ps} + d - d_{ps})}{f_{ps} jd_{ps}}$$

where:

$M^{+}$ = Positive moment evaluated from PCA publication
$A_s$ = Area of diaphragm ties
d = distance from extreme compressive fiber to the centroid of the diaphragm

$d_{ps}$ = distance from extreme compression fiber to the centroid of the strands
$jd_{ps}$ = internal moment arm, may be assumed as = $0.94d_{ps}$

Once the number of strands and embedment length have been determined, the section should be checked for ultimate capacity following the procedure presented in the referenced FHWA report.

$$M_u = \Phi \left[ A_{ps} f_{pu} \left( d_{ps} - \frac{a}{2} \right) + A_s f_y \left( d - \frac{a}{2} \right) \right]$$

$$f_{pu} = \frac{L_e - 8.25}{0.163}$$

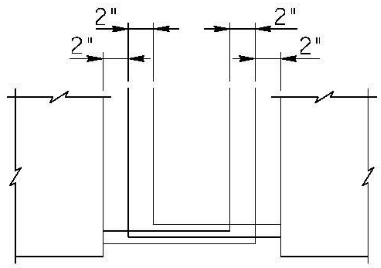
$$a = \frac{A_{ps} f_{pu} + A_s f_y}{0.85 f_e b}$$

where:

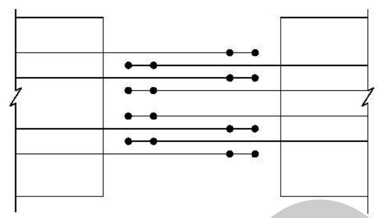
a = depth of compression stress block
b = width of member
$f_{pu}$ = stress of strands at general slip
$\Phi$ = workmanship factor, 0.90 for flexure

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. Refer to the figure below for strand pattern at girder end.

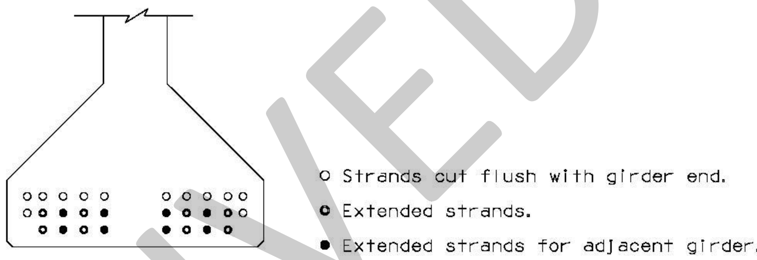
STRAND PATTERN AT GIRDER END.




ELEVATION



PLAN




TYPICAL SECTION

Method of Analysis for Precast I Girders

Section properties shall be based on 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.

AASHTO Type V modified and Type VI modified I-girders should be used in place of Type V and Type VI girders whenever possible.

The theoretical build-up depth shall be ignored for calculation of composite section properties.

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

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

5.14.1.5.1 Flange and Web Thickness

5.14.1.5.1a Top Flange

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

5.14.1.5.1b Bottom Flange

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

5.14.1.5.1c 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.

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 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. The design engineer may read and implement recommended design criteria included in the article "The Cause of Cracking in Post-Tensioned Concrete Bridges and Retrofit Procedures", by Walter Podolny, published in the PCI Journal, Vol. 30, No. 2, March-April 1985.

## 5.14.4 Slab Superstructures

### 5.14.4.1 Cast-in-Place Solid Slab Superstructures

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

### 5.14.4.2 Cast-in-Place Voided Slab Superstructures

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

### 5.14.4.3 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.

**Lateral Ties**
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.
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).

**Shear Keys**
After shear keys have been filled with an approved non-shrink mortar and reached a minimum strength of 5,000 psi, 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.

**Barriers**

Barriers shall have a ½ inch open joint at the midspan to prevent the barrier from acting as an edge beam and causing long-term differential deflection of the exterior beam.

5.14.4.3.3 Shear-Flexure Transfer Joints

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

**SECTION 6: STEEL STRUCTURES**

TABLE OF CONTENTS

## 6.1 Scope

This section contains guidelines to supplement provisions of Section 6 of the AASHTO LRFD Bridge Design Specifications for the analysis and design of steel components, splices and connections for beam and girder structures, frames, trusses and arches, as applicable. Metal deck systems in relation to steel stay-in-place formwork are covered in Section 9 of these guidelines.

Minimum vertical clearance for a bridge should be established based on future roadway configuration. For bridges spanning over railroads, minimum vertical clearance shall be based on the most recent railroad grade separation guidelines.

Design level load ratings of all bridges shall be performed per MBE (The Manual for Bridge Evaluation) latest edition. For new bridges the design Operating Load Rating (using HL93 live load) shall be 1.8 or more, unless approved by ADOT Bridge Group for bridges with specific circumstances. For widening of bridges the minimum design Operating Load Rating (using HL93 live load) shall be the Operating Load Rating (using HL93 live load) of the existing bridge or 1.5, whichever is greater. Coordination and approval from ADOT Bridge Group will be required in instances where these provisions cannot be met, for widening of existing bridges.

## 6.4 Materials

### 6.4.1 Structural Steels

Structural steel shall conform to the requirements specified in AASHTO LRFD Table 6.4.1-1, with the selection based on strength, serviceability and overall economy. All structural steels shall be ASTM A709 Grade 50 or 50W (AASHTO M270 Grade 50 or 50W).

ASTM A709 Grade 36 (AASHTO M270 Grade 36) steel may be used for miscellaneous applications, such as, anchor bolts, expansion joints, rods etc. All miscellaneous steel hardware exposed to weathering action shall be galvanized.

### 6.4.3 Bolts, Nuts and Washers

All structural fasteners shall be high-strength bolts, ASTM A325 (AASHTO M164). Type 1 bolts should be used with steels other than weathering steel. Type 3 bolts conforming to either ASTM A325 (AASHTO M164) or ASTM A490 (AASHTO M253) may be used with weathering steel. All washers, nuts, and bolts shall be galvanized. Type 1 bolts may be either hot-dip galvanized in accordance with ASTM A153 (AASHTO M232) Class C or mechanically galvanized in accordance with ASTM B695 (AASHTO M298) Class 50 with prior approval from ADOT Bridge Group.

### 6.4.7 Stainless Steel

The specifications of stainless steel material and their applications are listed in AASHTO LRFD Article 6.4.7. Stainless steel plate for use as a flat mating surface shall conform to AASHTO LRFD Article 14.7 as well as Section 14 of these guidelines.

6.4.8 Cables

Cables restrainers used to restrain a bridge under seismic loads shall conform to 3/4-inch diameter preformed, 6 x 19, wire strand core or independent wire rope core (IWRC). Cables shall be galvanized in accordance with the requirements in Federal Specification RR-W-410D and Right Regular Lay manufactured of improved plow steel with a minimum breaking strength of 42,000 pounds.

## 6.6 Fatigue and Fracture Considerations

6.6.1 Fatigue

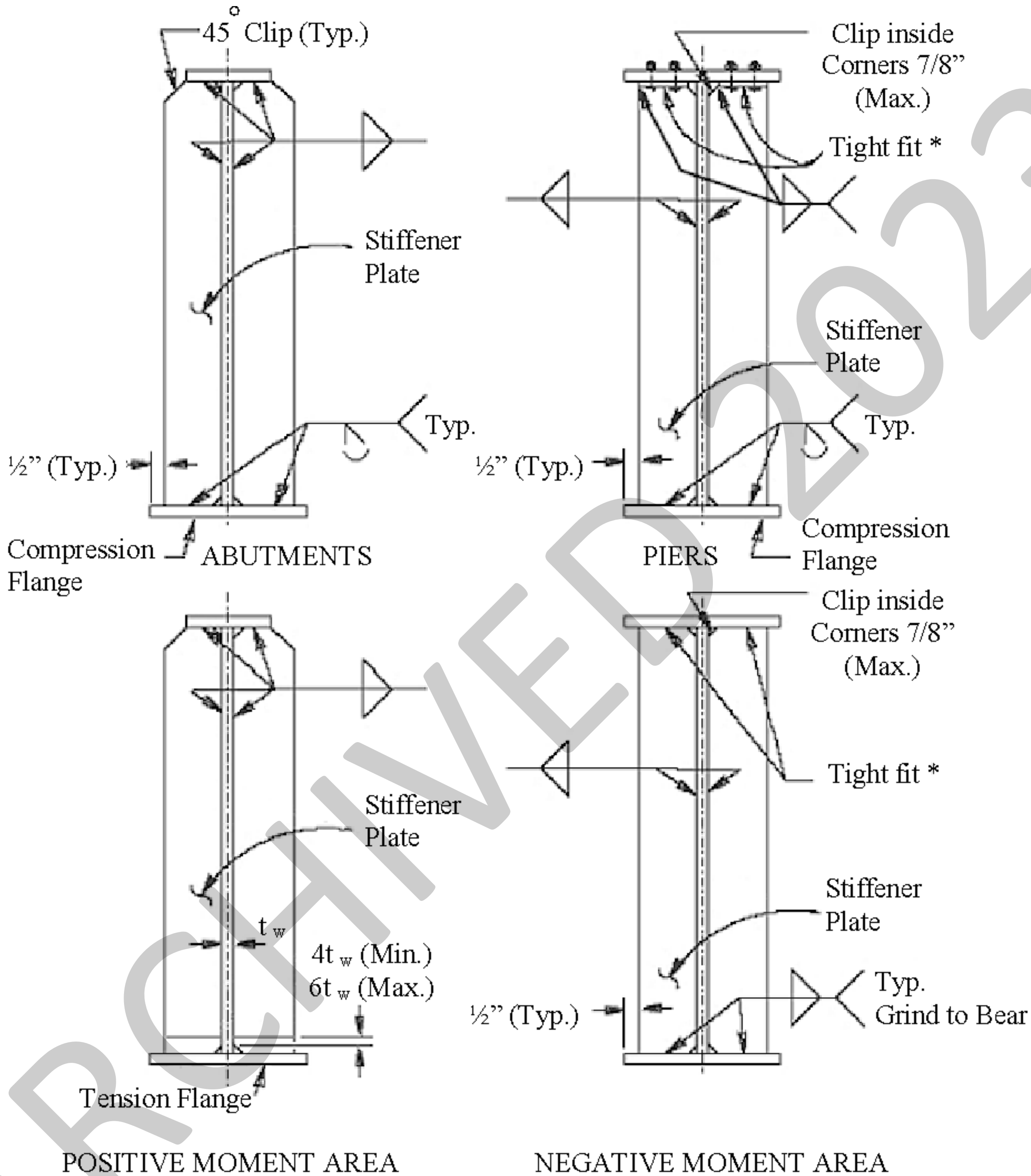
6.6.1.2 Load-Induced Fatigue

Structural members including splices, stiffeners, bracings, shear connectors, and fasteners, subjected to stress reversal due to applied live loads shall be designed, to limit stress due to fatigue, using welding detail categories A through C (refer to AASHTO LRFD Table 6.6.1.2.3-1). Welding detail category D and E shall not be used.

6.6.1.3 Distortion-Induced Fatigue

Transverse connection plates shall be connected to both the compression and tension flanges as stated in AASHTO LRFD Article 6.6.1.3.1. Structural bolts shall be used for plate connections to the tension flange. The following stiffener connection details should be followed.

FIGURE 1 – STIFFENER PLATE DETAILS




* See AASHTO LRFD Articles 6.6.1.3.1 and 6.10.11.1.1 for additional requirements.

### 6.6.2 Fracture

For Charpy V-notch testing, temperature zone levels shall be determined by the elevation of the structure location. The Charpy V-notch impact requirements for structural steel shall be for Temperature Zone 1 at elevations less than 6,000 feet and Temperature Zone 2 at elevations 6,000 feet and higher.

## 6.7 General Dimension and Detailing Requirements

### 6.7.2 Dead Load Camber

Steel structures shall be cambered during fabrication to compensate for non-composite and composite dead load deflections, and for vertical profile. Non-composite loads include weight of steel members and deck slab. Composite loads include weight of barrier, median, sidewalk etc. The future wearing surface shall not be included in the camber calculation. Camber information shall be provided on structural plans.

### 6.7.3 Minimum Thickness of Steel

Minimum thickness of steel shall conform to AASHTO LRFD 6.7.3 with the following exceptions:

- Structural steel including bracing, cross-frames, gusset plates, closed ribs in orthotropic decks, and fillers, shall not be less than 3/8-inch in thickness. The web thickness of rolled shape sections are exempted.
- Welded plate girder webs and flanges shall be sized in 1/8-inch increments. Webs shall not be less than 1/2-inch in thickness.

### 6.7.4 Diaphragms and Cross-Frames

Rolled beams and plate girders shall be provided with cross-frames or diaphragms at each support and with intermediate cross-frames or diaphragms placed in all bays, at intervals not to exceed 25-feet. Other design criteria and provisions for diaphragm and cross-frames shall conform to AASHTO LRFD Article 6.7.4. Flexibility of the bracing system should be evaluated to assure ductility of the diaphragms and cross frames. The stiffener plates, which also serve as connection plates, shall be placed parallel to the skew, for skew less than or equal to 20 degrees. Stiffener plates shall be placed normal to the web for skew greater than 20 degrees. Transverse intermediate stiffeners that are not connection plates shall be placed normal to the web.

**6.10 I-Section Flexural Members**

6.10.1 General

6.10.1.3 Hybrid Sections

Hybrid I-Section members shall not be used without the prior approval of ADOT Bridge Group.

6.10.10 Shear Connectors

Welded stud shear connectors shall be used and shall conform to AASHTO LRFD Article 6.4.4. Welded stud shear connectors shall be installed in the field to improve the safety of construction personnel. Channel shear connectors shall not be used.

6.10.11 Stiffeners

6.10.11.1 Transverse Intermediate Stiffeners

For exterior girders, transverse stiffeners shall be placed on the inside face only. Refer to Figure - 1 Stiffener Plate Details in Article 6.6.1.3 of these guidelines for detailing requirements.

6.10.11.2 Bearing Stiffeners

Each stiffener plate shall be attached to the compression flange by full penetration groove welds. Refer to Figure - 1 Stiffener Plate Details shown in Article 6.6.1.3 of these guidelines for detailing requirements.

6.10.11.3 Longitudinal Stiffeners

Longitudinal stiffeners shall not be used without prior approval of ADOT Bridge Group. Webs shall be sized to eliminate the need for longitudinal stiffeners.

6.10.12 Cover Plates

6.10.12.1 General

Welded cover plates shall be a minimum 1/2-inch narrower than the flange to which they are attached in order to accommodate a 1/4-inch fillet weld. Welded cover plates wider than the flange can contribute to a reduction in fatigue strength and shall not be used. Cover plate ends shall not be welded.

**6.11 Box Section Flexural Members**

Selection of these members requires ADOT Bridge Group approval.

## 6.13 Connections and Splices

### 6.13.3 Welded Connections

#### 6.13.3.1 General

In addition to AASHTO LRFD Article 6.13.3, all welding except for stud shear connectors shall be performed in the fabrication shop. With the exception of retrofit or repair work, no welding shall be performed in the field without prior approval of ADOT Bridge Group.

Provisions in AASHTO/AWS D1.5M/D1.5 Bridge Welding Code shall be followed to ensure appropriate information is provided in the contract documents to facilitate proper fabrication and quality control.

**SECTION 9: DECKS AND DECK SYSTEMS**

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**9.1 SCOPE**

This Section contains guidelines to supplement provisions of Section 9 of the AASHTO LRFD Bridge Design Specifications for the design of bridge decks and deck systems of reinforced concrete, prestressed concrete, metal, or various combinations thereof.

All design engineers are advised to review the example problems in Appendix – A of these guidelines for proper and correct application of various provisions of the AASHTO LRFD Specifications and these guidelines for design of bridge structural components.

**9.4 GENERAL DESIGN REQUIREMENTS**

Minimum concrete strength of bridge decks shall be, $f'_c$ = 4.5 ksi at 28 days.
Refer to Section 5, Article 5.4 Material Properties, of these guidelines for other requirements.

To provide protection against corrosion the minimum clear cover for reinforcing steel in new deck slabs shall be 2½ inch for top reinforcement and 1 inch for the bottom reinforcement.

Only #5 or #6 bar sizes shall be used as primary reinforcement in the transverse direction and shall be spaced at 1/2-inch increments. Minimum reinforcing spacing shall be 5 inches. Normally maximum transverse reinforcement spacing should not exceed 9 inches.

Bar sizes up to #11 may be used as primary reinforcement in the longitudinal direction in slab bridges. This also applies to continuity reinforcement over piers.

All new bridge construction located above an elevation of 4000 feet, or for areas where de-icing chemicals are used, deck slabs, barriers, anchor slabs and approach slabs reinforcing as well as portions of reinforcement projecting into the deck slabs shall be epoxy coated.

Deck protection systems shall be discussed in the Bridge Selection Report. Recommended options, other than epoxy coated reinforcing, shall be coordinated with ADOT Materials Group and shall be approved by ADOT Bridge Group.

For existing bridges, latex modified concrete overlay, silica-fume concrete overlay or a membrane system with a bonded wearing surface are alternate protection systems that may be considered. Implementation of either one of these alternatives requires coordination with ADOT Materials and Bridge Groups.

A 3/4" V-drip groove shall be located on the underside of the deck overhang for all bridges.

## 9.5 LIMIT STATES

9.5.2 Service Limit States

Deck design is controlled by Service Limit State I. The behavior of bridge decks shall be considered elastic. Decks shall be designed by the working stress method and as stated in this section.

Allowable tensile stress in reinforcing steel, $f_s$, shall be limited to 24 ksi.

## 9.6 ANALYSIS

9.6.1 Methods of Analysis

The most typical deck system used in Arizona is a cast-in-place deck slab spanning transversely over a series of girders. This type of deck shall be designed using an approximate elastic method and the criteria stated in this section.

Refined methods of analysis, such as the Finite Element Method, shall only be used for unconventional, complex structures and with prior approval from ADOT Bridge Group.

Dead load analysis shall be based on a strip method using the following simplified moment equation for both positive and negative moments:

$wS^2/10$, for deck slabs that are continuous over three spans or more
$wS^2/8$, for all other cases

where:

S = the effective span length specified in AASHTO LRFD Article 9.7.2.3
w = the uniformly distributed dead load of the slab system

The unfactored live load moments shall be obtained from AASHTO LRFD Section 4, Appendix A, Table A4-1. Negative moment values should be based on a distance of 0.0 inch from the centerline of girder to the design section.

## 9.7 CONCRETE DECK SLABS

9.7.1 General

9.7.1.1 Minimum Depth and Cover

The thickness of new deck slabs shall be designed in $^1/_2$" increments with the minimum thickness as follows:

| S (ft) | ≤ 7 | > 7 and ≤ 8.5 | > 8.5 and ≤ 10 | > 10 and ≤ 11.5 | > 11.5 and ≤ 13 |
|---|---|---|---|---|---|
| t (in) | 8.0 | 8.5 | 9.0 | 9.5 | 10.0 |

where:

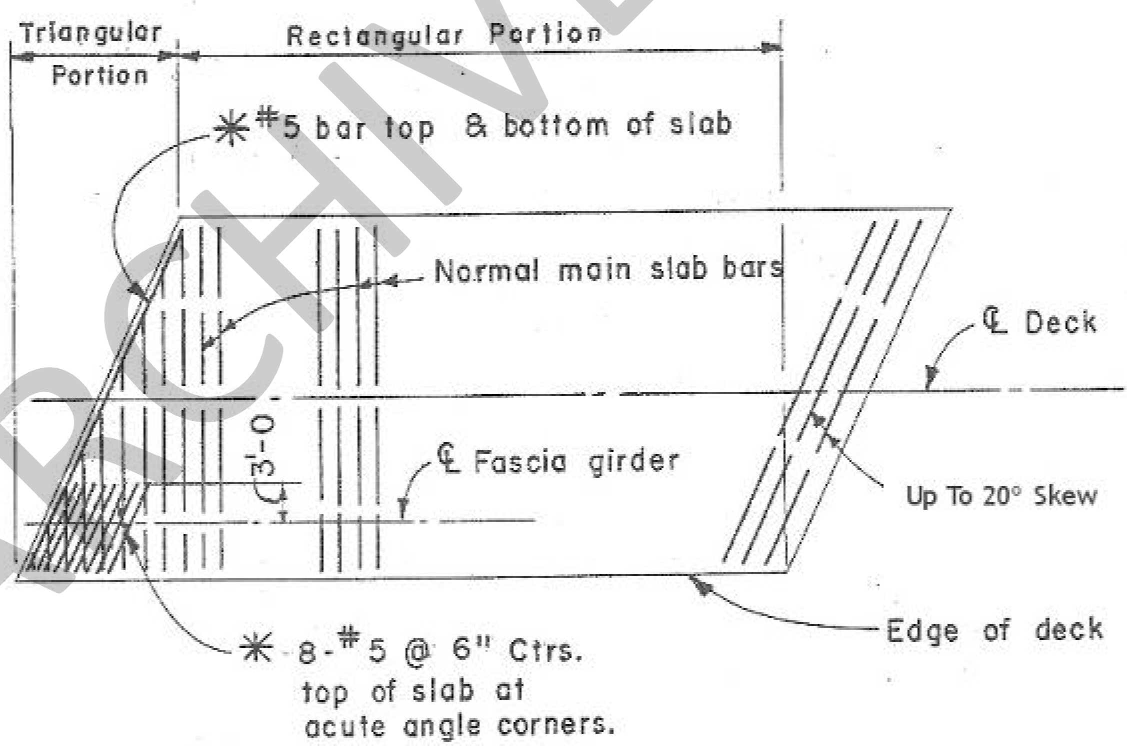
S = the effective span length specified in AASHTO LRFD Article 9.7.2.3
t = Minimum thickness of deck slab

Note that the slab thickness, t, includes $^1/_2$ inch wearing surface, which must be excluded for strength service analysis.

9.7.1.3 Skewed Decks

For a skew angle less than or equal to 20 degrees, the primary reinforcement shall be placed parallel to the skew. For skews greater than 20 degrees the reinforcing shall be placed perpendicular to the main supporting members. The effects of the skew shall be accounted for by providing additional short bars at the deck corners as shown in the figure below. Truss bars shall not be used.

Skewed Girder Bridges




* Use for Skews of 20° and more at each skewed corner.

9.7.3 Traditional Design

9.7.3.2 Distribution Reinforcement

Distribution reinforcement shall be calculated in accordance with AASHTO LRFD Article 9.7.3.2. The required reinforcement shall be placed in the secondary direction throughout the effective span length between girders in the bottom of the slab.

9.7.4 Stay-in-Place Formwork

9.7.4.1 General

Use of stay-in-place (SIP) formwork should be investigated for each bridge site during preliminary design and a discussion of this issue should be included in the Bridge Selection Report. Use of a SIP formwork system should be considered for the following situations:

When bridges span high traffic volume roadways, deep canyons, perennial streams or canals.
Where removal of conventional formwork would be difficult or hazardous.

When a SIP formwork system is selected, the contract documents shall include conceptual design and connection details for the SIP system. The contractor shall submit all SIP formwork design calculations and connection details to the design engineer for approval. Shop drawings for the girders including the location of inserts and the SIP formwork, shall be submitted concurrently for review and approval.

9.7.4.2 Steel Formwork

Steel formwork is the preferred stay-in-place formwork for bridge deck construction. The design engineer shall assume an additional 15 psf of dead load due to the weight of the forms and the concrete in the flutes. SIP formwork flutes shall only be filled with structural concrete, use of foam or polystyrene is not allowed.

Steel formwork shall not be considered to be composite with the concrete deck slab. The construction plans should state the assumed additional weight that the deck, girder and substructure have been designed for due to this method of construction.

If the SIP formwork is the only method of construction allowed the additional concrete in the steel flutes shall be included in the estimate of deck concrete quantities. If it is an option the additional quantity of concrete is not part of the deck concrete quantities and needs to be clearly stated in the plans.

The steel formwork shall be galvanized for corrosion protection.

9.7.4.3 Concrete Formwork

Precast stay-in-place concrete panels used with a cast-in-place concrete topping to create a final composite deck are another form of formwork for deck construction. The panels are designed to span transversely between the girders and are usually prestressed. Precast concrete deck panels are not recommended for SIP formwork due to the complexity of design and construction parameters such as:

Transfer and development lengths
Correct location of strands or reinforcing
Difficulty in providing for girder camber and proper seating of the panels
Longitudinal discontinuity resulting in possible reflective cracking at the ends of each panel
Difficulty in ensuring composite action with the cast-in-place concrete
Combined shrinkage and creep effects

Precast prestressed concrete deck panels may be considered for major or unusual girder bridges. A full discussion justifying the use of precast concrete deck panels, including all design and construction parameters, must be included in the Bridge Selection Report before final approval can be considered.

**DECK OVERHANG DESIGN**

The deck overhang shall be designed in accordance with AASHTO LRFD Section 13, Appendix A, Article A13.4. For Design Case 1, the deck shall be designed to resist both the axial force and the bending moments due to the dead load and the horizontal railings impact load. The vertical wheel load shall not be applied simultaneously with these loads. The net tensile strain in the extreme tension steel in the overhang reinforcing for Design Case 1, Extreme Event Load Combination II limit state, shall not exceed 0.025.

For Design Case 3 both the strength and service limit states shall be investigated. In the Service Limit State the design live load distribution shall be determined using Table 4.6.2.1.3-1.

When traffic barriers are located at the edge of the deck, the slab thickness of the overhang shall be at least 1 inch greater than the interior slab thickness. Deck reinforcement resisting overhang loadings shall be fully developed at the section under consideration. Reinforcing steel larger than #5 bars may require hooks at the edge of deck for development length.

Concrete barriers on continuous superstructures should have a $^1/_2$ inch open joint filled with bituminous joint filler located over piers. The joint should extend to within 8 inches of the deck surface with reinforcing below this level made continuous.

The values in the following table shall be used for the design of the deck overhang in conjunction with ADOT Bridge Group Standard Drawings (SD) for concrete barriers. Refer to the AASHTO LRFD Section 13, Appendix A, Article A13.3 for definition of the symbols contained in the table.

| Barrier Type | $M_b$ | $R_w$ | $M_c$ | $M_w$ | Top rail $M_p$ | Bot. rail $M_p$ | Post $M_p$ |
|---|---|---|---|---|---|---|---|
| SD 1.01 32" F shape | 0 | 58.83 kips | 6.17 [a] | 28.66 | -- | -- | -- |
| SD 1.02 42" F shape | 0 | 129.6 kips | 15.16 [b] | 56.42 | -- | -- | -- |
| SD 1.04 Parapet Rail [c] | 0 | -- | 12.04 | 30.15 | 12.65 | 12.65 [d] | 14.99 |
| SD 1.06 Two Tube Rail | -- | -- | -- | -- | 41.78 | 29.21 | 93.75 |

a. $M_c$ = 14.91 at open joints
b. $M_c$ = 24.24 at open joints
c. assumes 11-inch curb height at parapet
d. when fence is omitted

**RAILINGS**

New railing shall be designed in accordance with the latest AASHTO LRFD Specifications, Section 13 and these guidelines.

**SECTION 10: FOUNDATIONS**

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## 10.1 SCOPE

This Section contains guidelines to supplement provisions of Section 10 of the AASHTO LRFD Bridge Design Specifications for the analysis and design of foundations for highway structures. Provisions of this section shall apply to the design of spread footings, driven piles, and drilled shaft foundations.

Bridges that are designed based on soil-structure interaction principles shall not be used to span over roadways.

## 10.5 LIMIT STATES AND RESISTANCE FACTORS

10.5.2 Service Limit States

10.5.2.2 Tolerable Movements and Movement Criteria

Rotational movements shall be evaluated at the top of the substructure unit and at the deck elevation.

Tolerance of the superstructure to lateral movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects.

The bridge designer should limit the settlement of a foundation per 100 ft span to 0.75 in. Linear interpolation should be used for other span lengths. Higher settlements may be used when the superstructure is adequately designed for such settlements. Any settlement that is in excess of 4.0 in, including stage construction settlements if applicable, must be approved by the ADOT Bridge Group. The designer shall also check other factors, which may be adversely affected by foundation settlements, such as rideability, vertical clearance, and aesthetics.

Based on the ADOT Material Group's recommendation, the settlement, which shall be used for the structural evaluation of any span, shall be the larger of the two settlements at either end of that span assuming no settlement at the other end. Settlements shall be determined from the charts which are included in the Geotechnical Report based on service limit state. For settlements in excess of 0.75 in per 100 ft of span length, the superstructure shall be designed to sustain the forces induced due to such settlement. The bridge designer should use the full value of the settlement without deducting 0.75 in per 100 ft of span length. For bridges that will be built using a stage construction method, settlements that do not induce forces in the superstructure may be subtracted from settlements obtained from the Geotechnical Report when determining the value of the settlement to be used in designing the superstructure. For bridges involving complex stage construction, the bridge designer should coordinate with the geotechnical engineer when determining settlements.

The creep and the elasto-plastic characteristics of the soil surrounding the foundation, permit some moment relief for the columns. It is suggested that, for Service Limit State only, moments and shears due to prestressing could be reduced by 50%, and those due to thermal action could be reduced by 25%. These reductions are considered reasonable when applied to columns with a fixed connection to the foundation. When allowing limited foundation release using springs or some foundation translation, or if drilled shafts are being used, the prestress and thermal forces shall not be reduced. Therefore, reductions in the prestress and thermal forces must be used consistent with the analysis model.

10.5.5 Resistance Factors

10.5.5.2 Strength Limit States

10.5.5.2.4 Drilled Shafts

In general, a bridge abutment or pier foundation consisting of two or more drilled shafts is considered as a redundant foundation, unless the center-to-center spacing of the drilled shafts is six diameters or more.

Substructure systems spanning roadways, such as straddle bents, and supported by single drilled shafts at each end, shall not be considered redundant.

## 10.6 SPREAD FOOTINGS

10.6.1 General Considerations

10.6.1.1 General

Provisions of this Article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements.

In Arizona, the two most commonly used types of spread footings are:

- Isolated footings: which are used as individual support for the various parts of a substructure unit and may be stepped laterally.
- Combined footings: which are used to support more than one column for multi-column bents. They also could be used to support wall bents.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this Section.

Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

When sound soil materials exist near the surface, shallow foundations in the form of spread footings are commonly used. For foundation units situated in a stream, spread footings may be used when they can be placed on non-erodible rock. Spread footings used as bridge foundations shall not be supported by embankment fill material including embankments consisting of mechanically or otherwise stabilized earth systems.

The bridge designer shall size the footing to ensure that the limit bearing pressure and settlement will not be exceeded for any AASHTO LRFD group loading. The footing shall be properly designed to resist the maximum applied moments and shears.

Spread footings shall be designed for limit states and resistance factors as specified in AASHTO LRFD Article 10.5. Resistance factors for the strength limit state shall be taken as specified in AASHTO LRFD Table 10.5.5.2.2-1. Resistance factors for the service limit state shall be taken as 1.0.

Bridge designers shall include the following information per footing on the structure plan sheets:

- Total settlement which is used in the design of the footing based on the geotechnical report.
- The service limit state factored net bearing resistance (capacity) (in ksf) and the strength limit state factored net bearing resistance (capacity) (in ksf) which are used in the design of the footing based on the geotechnical report.

### 10.6.1.2 Bearing Depth

The depth of footing shall be determined in consideration of the character of the foundation materials and the possibility of undermining. Footings at stream crossings shall be founded at a depth of at least 2.0 ft. below the maximum anticipated depth of scour as determined by the ADOT Bridge Hydraulics section and ADOT Geotechnical Section.

The bottom of spread footings shall be set at least to the depth recommended in the bridge foundation report. The minimum top cover over the top of the footings shall be 1’-6. For footings located at elevations over 5,000 feet, the minimum depth of embedment to the bottom of footings shall be 6’-0 to prevent frost heave.

Consideration shall be given to the use of either a geotextile or graded granular filter layer to reduce susceptibility to piping in rip rap or abutment backfill.

### 10.6.6 Spread Footing Design Considerations

For the purpose of determining the bearing resistance, for both service limit state and strength limit state, the bridge designer shall provide the footing length (L) and depth of embedment ($D_f$) within 20 percent plus or minus to the geotechnical engineer. After receiving the above mentioned information from the bridge designer, the geotechnical engineer shall calculate the bearing resistance for the service limit state (for 0.25 inch, 0.50 inch, 0.75 inch, 1.00 inch, 1.50 inch and 2.00 inch settlements) and for the strength limit state. If more than 2 inch settlement is required, the bridge designer shall coordinate with the geotechnical engineer to obtain bearing resistance for those settlements for the service limit state.

The bridge designer will design the spread footing based on the bearing resistance provided by the geotechnical engineer and the memorandum “Development of Factored Bearing Resistance Chart by the Geotechnical Engineer for use by a Bridge Engineer to size spread footings on soils based on service and strength limit states”.

This memorandum is available on the ADOT Material Group website:

http://www.azdot.gov/Highways/Materials/Geotech_Design/Policy.asp.

## 10.7 DRIVEN PILES

### 10.7.1 General

#### 10.7.1.1 Application

Piling should be considered when spread footings cannot be founded on rock, or on competent soils at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, piles bearing on suitable materials below susceptible soils should be considered for use as a protection against these problems. Piles should also be considered where right-of-way or other space limitations would not allow the use of spread footings, or where removal of existing soil that is contaminated by hazardous materials for construction of shallow foundations is not desirable.

Piles should also be considered where an unacceptable amount of settlement of spread footings may occur.

Driven piles may be either H piles, pipe piles or prestressed concrete piles.

The geotechnical engineer is responsible for recommending when driven piles can be considered, the type of driven pile to be used, the service, strength or extreme event limit states capacity of the pile. The geotechnical engineer is also responsible for recommending the estimated pile tip elevation and any special requirements necessary to drive the piles. When steel piles are used, the corrosive life of the pile should be reported in the geotechnical report.

The bridge designer is responsible for ensuring that the axial capacity and the lateral capacity of the pile or pile group are not exceeded for any AASHTO LRFD limit states group loadings.

Driven piles could be classified as follows:

- Battered Pile: A pile driven at an inclined angle to provide higher resistance to lateral loads.

- Friction Pile: A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile.

- Point Bearing Pile: A pile whose support capacity is derived principally from the bearing resistance of the foundation material on which the pile tip rests.

- Combination Friction and Point Bearing Pile: A pile that derives its capacity from contribution of both friction resistance mobilized along the embedded pile and point bearing developed at the pile tip.

In Arizona, the following two types of piles are commonly used:

- Pipe Pile: 14 and 16 inch diameter steel pipes with 1/2 or 5/8 inch wall thickness are generally recommended for the shell. The pile is driven or vibrated down into the soil until the designed bearing capacity is reached. Then, the steel reinforcing cage is placed inside the shell. Finally, concrete is poured into the pipe. The bridge designer shall assume that the shell is not contributing to the structural capacity of the pile.

- Steel H-Pile: ASTM 709 grade 50 HP shape or as recommended by the geotechnical engineer shall be used. H-piles are generally classified as friction piles.

Bridge designers shall include the following information per abutment and pier foundation on the structure plan sheets:

- Total settlement which is used in the design of the driven pile based on the geotechnical report.
- Total unfactored axial load at the top of each driven pile before increasing the axial load to account for redundancy or group efficiency effects.
- Total unfactored axial load at the top of each driven pile which is used in the design of the pile after increasing the axial load for redundancy or group efficiency effects.

10.7.2 Service Limit State Design

10.7.2.2 Tolerable Movements

The requirements of AASHTO LRFD Article 10.5.2.1 and Article 10.5.2.2 of these guidelines shall apply.

## 10.8 DRILLED SHAFTS

### 10.8.1 General

#### 10.8.1.1 Scope

The provisions of this Section shall not be taken as applicable to drilled piles, e.g., augercast piles, installed with continuous flight augers that are concreted as the auger is being extracted.

All drilled shafts shall be constructed vertically. Battered drilled shafts are not allowed. The geotechnical engineer is responsible for recommending the minimum diameter of the shaft and for providing the necessary information for determining the minimum required embedment below a specified elevation to develop the required resistance to the design axial and lateral load. The geotechnical engineer is also responsible for determining the soil properties in each layer to be used in resisting lateral loads. In the Bridge Foundation Report, the geotechnical engineer shall specify a method of drilled shaft construction based on either dry or wet excavation. In the event of wet excavation, slurry, temporary casing, or permanent casing is usually recommended depending on the water table elevation and soil condition.

The axial and lateral capacity of the drilled shafts shall be reduced by ignoring the embedment within the specified scour depth as documented in the Bridge Hydraulics Report.

Drilled shafts installed under wet excavation conditions shall be inspected according to a method described in the ADOT Standard Specifications for Road and Bridge Construction or the project's special provisions. Two commonly used methods are the gamma-gamma logging device and the cross-hole sonic logging survey.

The following types of drilled shafts are commonly used in Arizona:

- Prismatic Shaft: A shaft with constant diameter throughout its entire length.

- Rock-Socketed Shaft: A shaft where its lower portion or its entire length is embedded into the rock strata. This type of drilled shaft requires special heavy duty drilling equipment. Where rock-socketed shafts require casing through the overburden soils, the socket diameter shall be at least 6.0 inch less than the inside diameter of the casing. For rock-socketed shafts not requiring casing through the overburden soils, the socket diameter may be equal to the shaft diameter through the soil. Unless otherwise specified by the geotechnical engineer, the minimum embedment into the rock strata shall be 10 feet. A separate pay item shall be set up to account for the rock socket.

- Bell Shaped Shaft: A shaft with a flared bell shape at its tip to increase the bearing area. This type of drilled shaft is more advantageous when stiff foundation material is documented which results in higher bearing capacity enabling the designer to reduce the drilled shaft length. The entire base area may be taken as effective in transferring the load only if appropriate provisions are included in the contract documents so that the bottom of the bell shaped drilled hole is cleaned and inspected prior to concrete placement. Due to the difficulty of properly cleaning the bottom of the hole, this type of drilled shaft is not a preferred alternative.

- Telescoping Shaft: A shaft with two or more segments of consecutively smaller diameters. In order to avoid using excessive steel casing for shoring purpose during drilled shaft construction when very loose foundation material is present, telescoping augering technique can be beneficial to accommodate varying soil conditions. The bridge designer should avoid using this type of drilled shaft due to the inherent difficulty of constructing it.

Bridge designers shall include the following information per drilled shaft on the structure plan sheets:

- Total settlement which is used in the design of the drilled shaft based on the geotechnical report.
- Total unfactored axial load at the top of the drilled shaft before increasing the axial load to account for redundancy or group efficiency effects.
- Total unfactored axial load at the top of the drilled shaft which is used in the design of the drilled shaft after increasing the axial load for redundancy or group efficiency effects.

#### 10.8.1.4 Battered Shafts

Battered shafts shall not be used. Where increased lateral resistance is needed, consideration should be given to increasing the shaft diameter or increasing the number of shafts.

### 10.8.2 Service Limit State Design

#### 10.8.2.1 Tolerable Movements

The requirements of AASHTO LRFD Article 10.5.2.1 and Article 10.5.2.2 of these guidelines shall apply.

#### 10.8.2.2 Settlement

##### 10.8.2.2.1 General

The settlement of a drilled shaft foundation involving either a single-drilled shaft or groups of drilled shafts shall not exceed the movement criteria selected in accordance with Article 10.5.2.2 of these guidelines.

If applicable, time-dependent and consolidation settlements, referred to as long-term settlements, of the drilled shaft foundation system shall also be determined by the geotechnical engineer. The bridge designer shall evaluate whether such settlements can be tolerated by the structure.

## 10.8.5 Drilled Shafts Design Considerations

The geotechnical engineer shall develop the following two charts:

- Chart 1: Strength axial resistance, plotted as abscissa, versus depth of embedment for various shaft diameters, plotted as ordinate.

- Chart 2: Service axial resistance for a given vertical displacement of the shaft top, plotted as abscissa, versus depth of embedment for various shaft diameters, plotted as ordinate. Chart 2 is repeated depending on the considered displacement values.

Note: Since project plans must show the elevations, the geotechnical engineer should indicate the elevation along with the depth on the ordinate axis.

The bridge designer shall use Chart 1 to evaluate the strength limit state and Chart 2 to evaluate the service limit state. In the event that the design displacement differs from the values provided in Chart 2, the bridge designer shall develop an additional Chart "Chart 3". This chart will display the developed axial resistance, on the ordinate axis, versus vertical displacement for a shaft of given diameter and depth of embedment, on the abscissa axis.

Note that Chart 3 is different from Chart 2 in the sense that Chart 3 is developed only for a specific diameter and depth of embedment while Chart 2 is developed for a range of shaft diameters and depths of embedment.

The bridge designer shall design the drilled shafts based on the memorandum "Development of Drilled Shaft Axial Resistance Charts for use by Bridge Engineers". This memorandum is available on the ADOT Material Group website:

http://www.azdot.gov/Highways/Materials/Geotech_Design/Policy.asp.

Unless otherwise specified in the bridge foundation report, the following criteria shall be used in designing drilled shaft foundations:

- Drilled shafts shall be designed for limit states and resistance factors as specified in AASHTO LRFD Article 10.5.
- All applicable service limit state load combinations in AASHTO LRFD Table 3.4.1-1 shall be used for evaluating lateral displacement of drilled shafts.
- Where soil deposit in which shafts have been installed is subject to settlement, due to consolidation or otherwise, in relation to the shafts, down drag loads shall be considered in the design of the drilled shafts.
- Drilled shafts shall be spaced a minimum of three diameters measured center-to-center of the shafts unless the geotechnical engineer approves a lower center-to-center spacing.
- Due to the fact that boulders may be encountered during the drilling operation, minimum diameter of the drilled shafts shall be four feet, unless the geotechnical engineer approves smaller diameter of drilled shafts for a specific site.

- Due to constructability issues, the length of a drilled shaft shall be limited to 20 times its diameter.
- Drilled shafts of six feet or more in diameter or which may be constructed using slurry or wet method, shall have 6 inches minimum clear cover of the reinforcements to the outside edge of the shaft.
- Drilled shafts of less than six feet in diameter and which are constructed in dry soil, shall have at least 3 inches minimum clear cover of the reinforcements to the outside edge of the shaft.
- Vertical reinforcing shall be detailed to provide the minimum recommended spacing in AASHTO LRFD Article 5.10.3. In no case the spacing between vertical reinforcing shall be less than 4 ½ inches.
- Horizontal ties shall be spaced not less than 6 inches and not more than 12 inches.
- Drilled shaft caps connecting two or more drilled shafts shall be sized along the length of the cap to extend a minimum of 9 inches from the edge of each exterior shaft.
- If wet excavation is anticipated for the drilling, inspection tubes shall be installed inside the drilled shaft for gamma-gamma logging device or cross hole sonic logging as per Special Provisions or current ADOT Standard Specifications for Road and Bridge Construction. The minimum number of inspection tubes shall be equal to the diameter of the drilled shaft, measured in feet, and rounded-up to the next whole integer, but not less than four. The inspection tubes shall be uniformly distributed along the inside circumference of the reinforcing steel cage.
- Confirmation shafts shall be designated in the bridge plan.
- If collapsing material or intermittent large boulders are found during the geotechnical investigation, a test drilled shaft may be constructed as part of the investigation and the results included in the final bridge foundation report.

## 10.10 GEOTECHNICAL REPORTS

### 10.10.1 General

Two geotechnical reports shall be prepared by the geotechnical engineer. The first one is a preliminary report that is used to start the foundation design resulting in stage II plans. The second one is the final geotechnical report which is used to document geotechnical parameters that are used in the final foundation design. The final geotechnical report shall be submitted part of stage II deliverables.

During the development of stage II plans, and especially while designing deep foundation elements, the bridge designer may discover the need for deeper borings than initially used in the preliminary geotechnical report. The bridge designer shall communicate this information to the geotechnical engineer so that the latter could base the final report on this additional data.

### 10.10.2 Preliminary Geotechnical Report

The preliminary geotechnical report shall contain the following:

- Introduction
- Site description
- Subsurface conditions
- Geomorphology of the waterway, if applicable
- Consideration of the effect of scour, if applicable
- Recommendation for test shafts, if necessary
- Soil parameters which are required for lateral and stability analysis of the foundation
- Foundation recommendations with force resistance charts such as axial and bearing as required for foundation design
- Boring log with general plan and elevation
- Laboratory test results

10.10.3 Final Geotechnical Report

The final geotechnical report shall contain the following additional items:

- Documentation of the test shaft results, if applicable
- Final foundation recommendations with force resistance charts such as axial and bearing as required for foundation design
- Special provisions, if necessary
- Foundation data sheets

**SECTION 13: RAILINGS**

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## 13.1 SCOPE

Bridge railing design shall be consistent with AASHTO LRFD Specifications Section 13. The design engineer is encouraged to use ADOT Bridge Group structure detail drawings wherever appropriate. Bridge Group website maintains the latest versions of these standard drawings. For convenience, links are provided in subsection 13.4 to all available ADOT railings structure detail drawings.

## 13.4 GENERAL

Bridge railing design for new bridges should be based on the current AASHTO LRFD Bridge Design Specifications for the selected Test Level.

All new bridge railings installed on the State Highway System should have a minimum of TL-4 rating. The preferred TL-4 bridge railing is the 34-inch F-shape bridge concrete barrier; and preferred TL-5 bridge railing is the 44-inch F-shape bridge concrete barrier. Other acceptable TL-4 and TL-5 bridge railings are available from ADOT Bridge Group.

44-inch F-shape bridge concrete barriers shall be used on directional ramps for freeway-to-freeway interchanges where ramps cross traffic lanes or highly occupied areas.

Bridge railings currently in use that have been found acceptable under the crash testing and acceptance criteria specified in NCHRP Report 230 will be considered as meeting the requirements of NCHRP Report 350 without the need of further testing.

For bridge modification considerations, existing bridge railings will normally be evaluated using AASHTO Standard Specifications for Highway Bridges and bridge railings replacements should be designed to either the AASHTO Standard Specifications or to the AASHTO LRFD Bridge Design Specifications, as appropriate on a case-by-case basis.

When sound walls are needed on a bridge, they should be placed behind bridge railings to maintain the intent of the design and to ensure that the railings will perform according to their crash test levels. A minimum gap of 2 inches should be maintained between the railings and the sound walls.

The following is a list of ADOT's railings structure detail drawings, method of measurement, and bid item numbers:

| Structure Detail Drawing | | Method of measurement | Bid Item Number |
|---|---|---|---|
| SD 1.01: 34-inch F-shape Bridge Concrete Barrier and Transition | | Linear Foot | 6011140 |
| SD 1.02: 44-inch F-shape Bridge Concrete Barrier and Transition | | Linear Foot | 6011141 |
| D 1.03: Thrie Beam Guard Rail Transition System | | Each | 9050430 |
| SD 1.04: Combination Pedestrian – Traffic Bridge Railing | | Linear Foot | 6011132 |
| SD 1.05: Pedestrian Fence for Bridge Railing SD 1.04 | | Linear Foot | 6011133 |
| SD 1.06: Two Tube Bridge Rail (4 sheets) | | Linear Foot | 6011134 |
| SD 1.11: Barrier Junction Box | Type I | Each | 7320475 |
| | Type II | Each | 7320476 |

Structure Detail Drawings are available on the Bridge Group website (click here).

Bridge concrete barriers and parapets shall not be constructed using slip forms. Painting the inside of bridge barriers should be avoided due to long-term maintenance concerns.

Rustication on the exterior of bridge barriers and parapets shall be limited to a thickness of 1 ½ in. Rustication may extend the full height of the barrier and parapet, excluding the 44-inch (nominal) F-shape bridge concrete barrier. The rustication height for 44-inch (nominal) F-shape barriers shall be limited to the bottom 32 inches, measured from the top of deck.