## 5.4.2 Normal and Structural Low-Density Concrete

### 5.4.2.1 Compressive Strength

Revise the 3rd paragraph as follows:

The specified compressive strength for prestressed concrete shall not be less than 4.0 ksi. The specified compressive strength for reinforced concrete shall not be less than 3.60 ksi.

## 5.5.3 Fatigue Limit State

### 5.5.3.1 General

Revise the 2nd Paragraph as follows:

In regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the F~~f~~atigue I load combination for infinite fatigue life as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.

Revise the 4th and 5th Paragraphs as follows:

Where consideration of fatigue is required, the stress range shall be determined using the F~~f~~atigue I load combination for infinite fatigue life as specified in Table 3.4.1-1.

The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads and prestress, and 1.75 times the fatigue load of Fatigue I is tensile and exceeds 0.095 $\sqrt{f'_c}$; otherwise, section properties for fatigue investigations shall be based on an uncracked section.

### C5.5.3.1

Revise the 2nd Paragraph as follows:

In determining the need to investigate fatigue, Table 3.4.1-1 specifies a load factor of 0.875 on the live load force effect resulting from the fatigue truck. The factor 2.0 specified in this Article is applied to the factored live load for a total of 1.750 times the unfactored force effect from the fatigue truck for infinite fatigue life.

### 5.5.3.2 Reinforcing Bars

Revise the 1st Paragraph as follows:

The stress range in straight reinforcement resulting from the F~~f~~atigue I load combination for infinite fatigue life, specified in Table 3.4.1-1, shall satisfy:

### 5.5.3.4 Welded or Mechanical Splices of Reinforcement

Revise the 1st Paragraph as follows:

For welded or mechanical connections that are subject to repetitive loads, the range of stress, $f_f$, resulting from the Fatigue I load combination for infinite fatigue life, and the Fatigue II load combination for finite fatigue life specified in Table 3.4.1-1, shall not exceed the nominal fatigue resistance given in Table 1.

Revise Table 5.5.3.4-1 as follows

**Table 5.5.3.4-1 Nominal Fatigue Resistance of Splices.**

| Type of Splice | $f_f$ for greater than 1,000,000 cycles |
|---|---|
| Grout-filled sleeve, with or without epoxy coated bar | 18 ksi |
| Cold-swaged coupling sleeves without threaded ends and with or without epoxy-coated bar; Integrally-forged coupler with upset NC threads; Steel sleeve with a wedge; One-piece taper-threaded coupler; and Single V-groove direct butt weld | 12 ksi |
| All other types of splices | 4 ksi |

Revise the 2nd Paragraph as follows:

Where the total cycles of loading, $N_{cyc}$, are less than 1 million, nominal fatigue resistance of splices specified in Table 1 ~~, $f_f$~~ may be increased by the quantity 24 (6–log$N_{cyc}$) ksi to a total not greater than the value ~~of $f_f$~~ given by the right side of Eq. 5.5.3.2-1 in Article 5.5.3.2. Higher values of nominal fatigue resistance ~~$f_f$~~, up to the value given by the right side of Eq. 5.5.3.2-1, may be used if justified by fatigue test data on splices that are the same as those that will be placed in service.

**5.5.4 Strength Limit State**
**5.5.4.1 General**
**5.5.4.2 Resistance Factors**

*5.5.4.2.1 Conventional Construction*

Add a new 2nd "bullet" as follows:

- For tension-controlled cast-in-place prestressed concrete sections as defined in Article 5.7.2.1……………………..…..0.95

*C5.5.4.2.1*

Delete Fig. C5.5.4.2.1-1 and replace with the following:

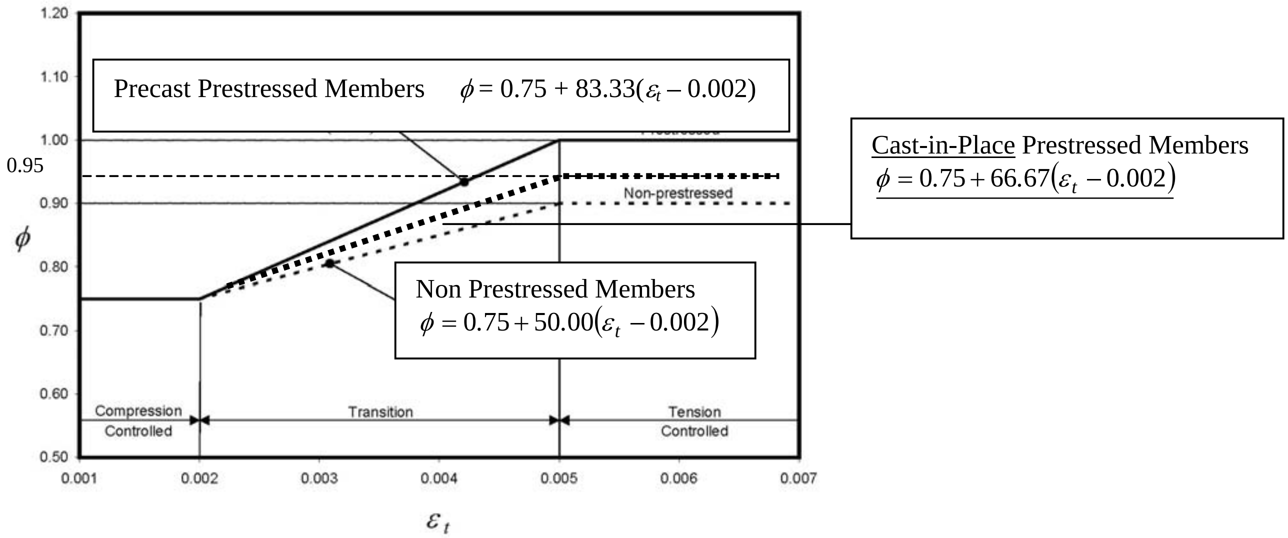


**Figure C5.5.4.2.1-1 – Variation of $\phi$ with net tensile strain $\varepsilon_t$ for Grade 60 reinforcement and for prestressed members.**

**5.5.5. Extreme Event Limit State**

Revise as follows:

The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1, as may be appropriate to its site and use. Resistance factors shall be 1.0.

### 5.7.2.1 General

Revise the 11th "bullet" as follows:

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. For non-prestressed concrete flexural sections including girders, bent caps, and deck slabs, the net-tensile strain in the extreme tension steel shall not be less than 0.004.

Add a new "bullet" as follows:

- In the approximate flexural resistance equations of Articles 5.7.3.1 and 5.7.3.2, $f_y$ and $f'_y$ may replace $f_s$ and $f'_s$, respectively, subject to the following conditions:
  - $f_y$ may replace $f_s$ when, using $f_y$ in the calculation, the resulting ratio $c/d_s$ does not exceed 0.6. If $c/d_s$ exceeds 0.6, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.
  - $f'_y$ may replace $f'_s$ when, using $f'_y$ in the calculation, $c \geq 3d'_s$. If $c < 3d'_s$, the compression reinforcement may be conservatively ignored ($A'_s = 0$), or strain compatibility shall be used to determine the stress in the mild steel compression reinforcement.

### C5.7.2.1

Revise the 4th paragraph as follows:

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Members, such as girders in the superstructure, bent caps beams and deck slabs are usually flexural members. When such members do not have any prestressing, then their sections are designed as tension controlled sections. Compression members such as columns and shafts are usually compression controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region….
*(rest of text remains the same)*

Add new paragraphs as follows:

When using the approximate flexural resistance equations in Articles 5.7.3.1 and 5.7.3.2, it is important to use the appropriate stresses in both the tension and compression mild steel reinforcement to obtain accurate results. In previous editions of *AASHTO LRFD Bridge Design Specifications*, the maximum reinforcement limit of $c/d_e \leq 0.42$ assured that the mild tension steel would yield at nominal flexural resistance, but this limit was eliminated in the 2006 Interim revisions. The current limit of $c/d_s \leq 0.6$ assures that the mild tension steel will be at or near yield, while $c \geq 3d'_s$ assures that the mild compression steel will yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

The mild steel tension reinforcement limitation does not apply to prestressing steel used as tension reinforcement. The equations used to determine the stress in the prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis.

*5.7.3.1.1 Components with Bonded Tendons*

Delete Eq. 5.7.3.1.1-3 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{f_s} - A'_s \underline{f'_s} - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Delete Eq. 5.7.3.1.1-4 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{f_s} - A'_s \underline{f'_s}}{0.85 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Delete the definitions for $f_y$ and $f'_y$ and replace with the following:

<u>$f_s$ = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>

<u>$f'_s$ = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>

Delete the last paragraph of Article 5.7.3.1.1:

~~The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of $f'_y$.~~

*5.7.3.1.2 Components with Unbonded Tendons*

Delete Eq. 5.7.3.1.2-3 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{f_s} - A'_s \underline{f'_s} - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w}$$

Delete Eq. 5.7.3.1.2-4 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{f_s} - A'_s \underline{f'_s}}{0.85 f'_c \beta_1 b}$$

Delete the last paragraph:

~~The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of $f'_y$.~~

*5.7.3.2.2 Flanged Sections*
Delete Eq. 5.7.3.2.2-1 and replace with the following:

$$\cancel{M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s' f'_y\left(d_s' - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)}$$

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s \underline{f_s}\left(d_s - \frac{a}{2}\right) - A_s' \underline{f'_s}\left(d_s' - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Delete the definitions for $f_y$ and $f'_y$ and replace with following:

<u>$f_s$ = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>
<u>$f'_s$ = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>

**5.7.3.4 Control of Cracking by Distribution of Reinforcement**

Delete Eq. 5.7.3.4-1 and replace as follows:

$$\cancel{s \leq \frac{700\gamma_c}{\beta_s f_s} - 2d_c}$$

$$s \leq \frac{700\gamma_c}{\beta_s \underline{f_{ss}}} - 2d_c \qquad (5.7.3.4\text{-}1)$$

Revise the 3rd and 4th paragraphs as follows:

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearances and/or corrosion. Class 2 exposure condition applies ~~to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and~~ when there is increased concern of appearance and/or corrosion. <u>Class 2 exposure condition also applies to all bridge decks</u>.

In the computation of $d_c$, the actual concrete cover thickness is to be used <u>except that in the case of bridge decks supported on girders, the clear cover to the top transverse deck reinforcing bars shall be taken as 2.5 inches to compute $d_c$ in Eq. 1</u>.

*5.7.3.6.2 Deflection and Camber*

Revise the 1st paragraph and add a 2nd paragraph as follows:

Instantaneous d~~D~~eflection ~~and camber~~ calculations shall consider appropriate combinations of dead load, live load, prestressing forces, erection loads, concrete creep and shrinkage, and steel relaxation.

Long-term deflection calculations to estimate camber shall consider deflections due to appropriate combinations of all the above mentioned load effects except for those due to live load.

Delete the 5th paragraph and replace with the following:

~~Unless a more exact determination is made, the long-time deflection may be taken as the instantaneous deflection multiplied by the following factor:~~

- ~~If the instantaneous deflection is based on~~ ~~$I_g$: 4.0~~
- ~~If the instantaneous deflection is based on~~ ~~$I_e$: $3.0\text{-}1.2(A'_s/A_s) >= 1.6$~~

Long-term deflection of cast-in-place structures may be calculated by multiplying the instantaneous deflection values based on $I_g$ with the following factors:

- For nonprestressed concrete structures: 4.0
- For prestressed concrete structures: 3.0

Alternatively, long-term deflection of cast-in-place non-prestressed concrete structures may be calculated by multiplying the instantaneous deflection values based on $I_e$ with the following factor:

$$3.0 - 1.2(A'_g/A_s) \geq 1.6 \quad (5.7.3.6.2\text{-}3)$$

where:

$A'_s$ = area of compression reinforcement (in²)

$A_s$ = area of nonprestressed tension reinforcement (in²)

*C5.7.3.6.2*

Revise the 1st paragraph as follows:

"Camber" is the deflection built into a member, other than by prestressing, in order to achieve a desired grade. For structures such as segmentally constructed bridges, camber calculations should be based on the modulus of elasticity and the maturity of the concrete when each increment of load is added or removed, as specified in Articles 5.4.2.3 and 5.14.2.3.6.

Add a new 2nd paragraph as follows:

Past experiences with cast-in-place box girder bridges show that the design predictions of camber based on $I_g$ are generally in conformance with field-measured values.

Revise the last paragraph as follows:

In prestressed concrete, the long-term deflection ~~is usually~~ may be based on mix-specific data where available, possibly in combination with the calculation procedures in Article 5.4.2.3. Other methods of calculating deflections which consider the different types of loads and the sections to which they are applied, such as that found in (*PCI 1992*), may also be used.

### 5.8.2.1 General

Revise the 3rd paragraph as follows:

~~The equivalent factored shear force, $V_u$, shall be taken equal to~~: The following shall be satisfied:

For solid sections:

$$\sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \leq V_r \qquad 5.8.2.1\text{-}6$$

where:

$p_h$ = perimeter of the centerline of the closed transverse reinforcement (in.)

$T_u$ = factored torsional moment (kip-in.)

For box sections:

$$V_u + \frac{T_u d_s}{2A_o} < V_r \qquad 5.8.2.1\text{-}7$$

~~where:~~

~~ph = perimeter of ......~~

$T_u$ = factored torsional moment (kip-in.)

### C5.8.2.1

Revise the 7th paragraph as follows:

In box girders, torsion introduces shear forces in the webs as well as in the top and bottom slab. In most box girder sections, the torsional shear in interior girder webs will be negligible and is primarily resisted by exterior girders. ~~For a box girder, t~~The shear flow due to torsion is added to the shear flow due to flexure in one exterior web and subtracted from shear flow due to flexure in the opposite web. In the controlling web, the second term in Eq. 7 comes from integrating the distance from the centroid of the section to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying ~~by~~ the shear flow by the web height measured between the shear flow paths in the top and bottom slabs. This height may be conservatively taken as *h* since calculating the value of $d_s$ would be ~~which has a value approximately equal to that of $d_s$,~~ cumbersome. If the exterior girder is sloped this distance should be divided by the sine of the web angle from the horizontal.

### 5.8.2.9 Shear Stress on Concrete

Revise the 2nd paragraph as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width. It is not necessary to reduce $b_v$ for the presence of ducts in fully grouted cast-in-place box girder frames.

### C5.8.2.9

Replace the 1st paragraph and Eq. 1with the following:

~~For flexural members complying with Eq. 5.7.3.3.1-1, the distance between the resultants of the tensile and compressive forces due to flexure can be determined as:~~

$$\cancel{d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}}}$$

The effective depth from extreme compression fiber to the centroid of tensile force in the tensile reinforcement can be determined as:

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_s d_s}{A_{ps} f_{ps} + A_s f_s} \qquad C5.8.2.9\text{-}1$$

### 5.9.3 Stress Limitations for Prestressing Tendons

Revise Table 5.9.3-1 as follows:

| | | | |
|---|---|---|---|
| ~~Prior to Seating~~ | 0.90$f_{py}$ | ~~0.90fpy~~ | 0.90$f_{py}$ |

| | | | |
|---|---|---|---|
| Maximum Jacking Stress | 0.90$f_{py}$ | 0.75$f_{pu}$ (see note) | 0.90$f_{py}$ |

Add a note below Table 5.9.3-1 as follows:

Note: For longer frame structures, tensioning to 0.90$f_{py}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value (low relaxation strand, only).

*5.9.4.2.2 Tension Stresses*

Revise Table 5.9.4.2.2-1 as follows.

**Table 5.9.4.2.2-1 Tensile Stress Limits in Prestressed concrete at Service Limit State After Losses, Fully Prestressed Components**

| Bridge Type | Location | Stress Limit |
|---|---|---|
| All Prestressed Bridges | Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections | |
| | • For components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only. | No tension |
| Other Than Segmentally Constructed Bridges | Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections | |
| | • For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and are located in Caltrans' Environmental Areas I or II. | $0.19\sqrt{f'_c}$ (ksi) |
| | • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans' Environmental Area III. | $0.0948\sqrt{f'_c}$ (ksi) |
| | • For components with unbonded prestressing tendons. | No tension |
| Segmentally Constructed Bridges | Longitudinal Stresses Through Joints in the Precompressed Tensile Zone | |
| | • Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 $f_y$; internal tendons or external tendons | $0.0948\sqrt{f'_c}$ (ksi) |
| | • Joints without the minimum bonded auxiliary reinforcement through joints | No tension |
| | Transverse Stresses Through Joints | |
| | • Tension in the transverse direction in precompressed tensile zone | $0.0948\sqrt{f'_c}$ (ksi) |
| | Stresses in Other Areas | |
| | • For areas without bonded reinforcement | No tension |
| | • In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 $f_y$, not to exceed 30 ksi. | $0.19\sqrt{f'_c}$ (ksi) |
| | Principal Tensile Stress at Neutral Axis in Web | |
| | • All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures. | $0.110\sqrt{f'_c}$ (ksi) |

*5.9.5.2.2b Post-Tensioned Construction*

Revise Table 5.9.5.2.2b-1 as follows:

**Table 5.9.5.2.2b-1 Friction Coefficients for Post-Tensioning Tendons**

| Type of Steel | Type of Duct | K (1/ft) | $\mu$ |
|---|---|---|---|
| Wire or strand | Rigid and semi-rigid galvanized metal sheathing<br>Tendon Length: | | ~~0.15-0.25~~ |
| | ≤ 600 ft | 0.0002 | 0.15 |
| | 600 ft ≤ 900 ft | 0.0002 | 0.20 |
| | 900 ft ≤ 1200 ft | 0.0002 | 0.25 |
| | > 1200 ft | 0.0002 | >0.25 |
| | Polyethylene | 0.0002 | 0.23 |
| | Rigid steel pipe deviators for external tendons | 0.0002 | 0.25 |
| HS bars | Galvanized metal sheathing | 0.0002 | 0.30 |

*C5.9.5.2.2b*

Add a new last paragraph:

For tendon lengths greater than 1200 feet, investigation is warranted on current field data of similar length frame bridges for appropriate values of $\mu$.

*5.9.5.2.3 Elastic Shortening*

*5.9.5.2.3b Post-Tensioned Members*

Delete Eq. 5.9.5.2.3b-1 and replace with the following:

$$\cancel{\Delta f_{pES} = 0.50 \frac{N-1}{2N} \frac{E_p}{E_{ci}} f_{cgp}}$$

$$\Delta f_{pES} = 0.50 \frac{E_p}{E_{ci}} f_{cgp}$$

*C5.9.5.2.3b*

Delete Eq. C5.9.5.2.3b-1 and replace with the following:

$$\cancel{\Delta f_{pES} = 0.50 \frac{N-1}{2N} \frac{A_{ps} f_{pbt} \left(I_g + e_m^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}}$$

$$\Delta f_{pES} = 0.50 \frac{A_{ps} f_{pbt} \left(I_g + e_m^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}$$

**5.9.5.3 Approximate Estimate of Time-Dependent Losses**

Add a new last paragraph:

For post-tensioned members, the approximate estimate of time-dependent losses may be taken as the lump sum value of 25 ksi.

**C5.9.5.3**

Add a new last paragraph:

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretentioned members and should not be used for post-tensioned structures. Preliminary research at UCSD indicates that the time-dependent losses for cast-in-place post-tensioned structures are between 25 ksi and 30 ksi. Until the research is completed, and, in lieu of a more detailed analysis, a lump sum value for losses in post-tensioned members is provided.

**Table 5.9.5.3-1 Time-Dependent Losses in ksi.**

| Type of Beam Section | Level | For Wires and Strands with $f_{pu}$ = 235, 250 or 270 ksi | For Bars with $f_{pu}$ = 145 or 160 ksi |
|---|---|---|---|
| Rectangular Beams, Solid Slab | Upper Bound<br>Average | 29.0 + 4.0 *PPR*<br>26.0 + 4.0 *PPR* | 19.0 + 6.0 *PPR* |
| Pretensioned Box Girder | Upper Bound<br>Average | 21.0 + 4.0 *PPR*<br>19.0 + 4.0 *PPR* | 15.0 |
| Single T, Double T, Hollow Core and Voided Slab | Upper Bound<br>Average | $39.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$<br>$33.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$ | $31.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$ |

### 5.11.4.3 Partially Debonded Strands

Revise the 2nd and 3rd paragraphs as follows:

The number of partially debonded strands should not exceed ~~25~~ 33 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed ~~40~~ 50 percent of the strands in that row.

**5.12.3 Concrete Cover**

Delete the existing text and table, and replace with the following:

The minimum concrete cover for protection of reinforcement against corrosion due to chlorides shall be as provided in Table 5.12.3-1.

"Corrosive" water or soil contains more than 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than 2,000 ppm and/or a pH of less than 5.5 shall be considered non-corrosive in determining minimum cover from Table 5.12.3-1, but shall conform to the requirements of Article 5.12.5.

Marine atmosphere includes both the atmosphere over land within 1,000 feet of ocean or tidal water, and the atmosphere above the splash zone. Tidal water, from corrosion considerations, is any body of water having a chloride content greater than or equal to 500 ppm. The splash zone is defined as the region from the Mean Lower Low Water (MLLW) elevation to 20 feet above the Mean Higher High Water (MHHW) elevation and/or a horizontal distance of 20 ft. from the edge of water. The concrete cover in structural elements that are in direct contact with ocean spray shall be based on the requirements for a chloride concentration greater than 10,000 ppm in the corrosive splash zone.

C5.12.3

Delete the existing text, and replace with the following:

The table for minimum concrete cover for protection against corrosion has been developed for a 75-year design life. However, the service life of bridge decks and barrier rails are typically less than 75 years. Therefore, the concrete mix design and cover requirements for corrosion protection of decks and barrier rails have incorporated these aspects.

Environmental conditions such as proximity to corrosive atmosphere, marine environment, wave action, water table elevation and chloride content have been incorporated in determining the cover requirements.

Corrosion protection can be improved by increasing concrete denseness or imperviousness to water, as well as by furnishing other protection methods. Such methods include:

a) a reduction in water-to-cementitious material ratio;
b) incorporating mineral admixtures/ supplementary cementitious materials into concrete mix design.
c) use of different kinds of epoxy-coated reinforcing bars (ECR);
d) protective concrete coatings;
e) use of chemical admixtures;
f) cathodic protection, and,
g) use of alternate materials.

The minimum concrete cover, concrete mix and epoxy-coated reinforcement requirements for structural elements exposed to deicing salt, snow run-off or snow blower spray shall be adopted only if the Engineer determines that the structural elements are directly exposed to these corrosive conditions. For example, when the deck is subjected to deicing salt, snow run-off or snow blower spray, it is unlikely that the girders or bent cap will be exposed to the same harsh condition, particularly when there are no deck joints. Therefore, the girders and the bent cap may be designed for a non-corrosive exposure condition.

If other considerations, such as a need to reduce the dead load of a structure, require a further reduction in concrete cover than those specified in Table 5.12.3-1, then a reduction in cover should only be done after a thorough investigation and research into existing state-of-practice.

## Table 5.12.3-1 Minimum Concrete Cover (inches) for 75 – year Design Life

| | Exposure condition | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Non-corrosive Atmosphere/ soil/ water | Marine Atmosphere | Corrosive soil above MLLW level | | | Corrosive soil below MLLW level | Corrosive water permanently below MLLW level | Corrosive splash zone | | | Deicing salt, snow run-off, or snow blower spray (a), (c),(e) |
| | | | Chloride Concentration (ppm) | | | | | Chloride concentration (ppm) | | | |
| | | | 500-5,000 (a) | 5,001-10,000 (a) | Greater than 10,000 (a) | (a) | (a), (b) | 500-5,000 (a),(b) | 5,001-10,000 (a),(b) | Greater than 10,000 (a),(b) | |
| Footings & pile caps | 3 | 3 | 3 | 4 | 5 | 3 | 2 | 2 | 3 | 3.5 | 2.5 |
| Walls, columns & cast-in-place piles | 2 | 3 | 3 | 4 | 5 | 3 | 2 | 2 | 3 | 3.5 | 2.5 |
| Precast piles and pile extensions | 2 | 2[d] | 2[d] | 2[b],[d] | 3[b],[d] | 2[d] | 2 | 2 | 2[d] | 2.5[d] | 2[d] |
| Top surface of deck slabs | 2 | 2.5 | | | | | | 2.5 | 2.5 | 2.5[d] | 2.5 |
| Bottom surface of deck slab[g] | 1.5 | 1.5 | | | | | | 2 | 2.5 | 2.5[d] | 2.5 |
| Bottom slab of box girders | 1.5 | 1.5 | | | | | | 2 | 2.5 | 2.5[d] | 1.5 |
| Cast-in-place "I" and "T" girders; exposed faces of box-girder webs, bent caps, diaphragms, and hinged joints (f) | 1.5 | 3 | | | | | | 2 | 2.5 | 2.5[d] | 3 |
| Curbs & railings | 1 | 1[b] | | | | | | 1 | 1 | 1[d] | 1 |
| Concrete surface not exposed to weather, soil or water | Principal reinforcement; 1.5 inches<br>Stirrups, ties and spirals: 1.0 inch | | | | | | | | | | |

**General Notes:**

1. Mineral admixtures / supplementary cementitious materials are required for all exposure conditions, except for 'non-corrosive' exposure conditions.
2. For protection of bundled bars, prestressing steel and /or ducts, see Articles 5.12.3-1, 5.12.3-2 and 5.12.3-3.
3. The minimum cover at the corners, beveled edges, and curved surfaces shall be the same as that in the corresponding structural elements.

**Footnotes:**

(a) The maximum water to cementitious material ratio shall not exceed 0.40.
(b) Use pre-fabricated epoxy coated reinforcing bars (ECR).
(c) Use post-fabricated ECR.
(d) Mineral admixtures / supplementary cementitious materials, in addition to those required by General Notes (1), may be required.
(e) The minimum concrete cover and other requirements in structural elements exposed to de-icing salt, snow run-off, or snow blower spray shall be adopted only where the structural elements are directly exposed to these corrosive conditions, otherwise the requirements specified for non-corrosive conditions shall be adopted.
(f) For precast "I" and "T" girders, the minimum cover may be reduced (depending on site conditions).
(g) Permanent support bars placed in the bottom of the deck slab may have a cover that is ½ inch less than that shown in the table.

### 5.12.3.1 Protection for Bundled Bars

For bundled bars, the minimum concrete cover in non-corrosive atmosphere shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches; except for concrete cast against and permanently exposed to non-corrosive soil, where the minimum cover shall be 3 inches. In corrosive environment, the cover shall be the same as that specified in Table 5.12.3-1, except that it shall not be less than the cover specified for bundled bars in non-corrosive environment.

### 5.12.3.2 Protection for Prestressing Tendons

In corrosive environments, the minimum concrete cover to prestressing steel not placed within ducts, shall be the same as that specified for reinforcement (Table 5.12.3-1), except that when epoxy-coated reinforcement is required per Table 5.12.3-1, the prestressing steel shall either be epoxy-coated or the minimum concrete cover to the prestressing steel shall be increased by 1.0 inch beyond that specified in Table 5.12.3-1.

Ducts for internal post-tensioned tendons, designed to provide bonded resistance, shall be grouted after stressing.

Other tendons shall be permanently protected against corrosion and the details of protection shall be indicated in the contract documents.

### C5.12.3.2

In certain cases, such as the tieing together of longitudinal precast elements by transverse post-tensioning, the integrity of the structure does not depend on the bonded resistance of the tendons, but rather on the confinement provided by the prestressing elements. The unbonded tendons can be more readily inspected and replaced, one at a time, if so required.

External tendons have been successfully protected by cement grout in polyethylene or metal tubing. Tendons have also been protected by heavy grease or other anticorrosion medium where future replacement is envisioned. Tendon anchorage regions should be protected by encapsulation or other effective means. This is critical in unbonded tendons because any failure of the anchorage can release the entire tendon.

### 5.12.3.3 Protection for Ducts

The minimum concrete cover for protection of ducts in corrosive environment shall be the same as that specified for reinforcement in Table 5.12.3-1, except that:

(a) the concrete cover over the duct shall not be less than one-half the diameter of the duct; and,

(b) when epoxy-coated reinforcement is required, the minimum concrete cover over the duct shall be increased by 0.50 inches beyond that specified for reinforcement in Table 5.12.3-1, but shall not be less than that specified in (a).

**5.12.4 Protection Against Acids and Sulfates**
Delete and replace with the following:

Exposed reinforcement, inserts and plates that are either attached to concrete or will be bonding with concrete, as well as other ferrous hardware, attachments and installations shall be properly protected from corrosion in accordance with the requirements of Table 5.12.3-1. Hot-dip galvanizing or an equivalent protective method may also be used. Appropriate reductions in requirements are permitted depending on the exposure conditions and/or duration.

**5.12.5 Protection Against Acids and Sulfate**
Delete and replace with the following:

The durability of concrete may be adversely affected by contact with acids and sulfates present in soil or water. When concrete is exposed to an acidic and/or a sulfate environment, then a special concrete mix design is required.

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