5.3 — NOTATION

Revise the definition of A_ℓ:

A_ℓ = area of longitudinal torsion reinforcement in ~~the exterior web of the~~ a box girder (in.^{2}); area of longitudinal column reinforcement (in.^{2}) (5.8.3.6.3) (5.11.5.2.1)

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5.3 — NOTATION

Revise the following definition:

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses), not including the effects of secondary moment, at extreme fiber of section where tensile stress is caused by externally applied loads (ksi) (5.7.3.3.2)

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5.4.2.1 — Compressive Strength

Revise the 3rd paragraph as follows:

The specified compressive strength for prestressed concrete ~~and decks~~ shall not be less than 4.0 ksi. The specified compressive concrete strength shall not be less than 3.6 ksi for reinforced concrete.

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5.4.6.2 — Size of Ducts

Modify the 2nd Paragraph as follows:

The size of ducts shall not exceed ~~0.4~~ 0.5 times the least gross concrete thickness at the duct.

5.5.3.1 — General

Revise the 2nd Paragraph of Article as follows:

In regions of compressive stress due to unfactored permanent loads and prestress in reinforced concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.

C5.5.3.1

Revise the 3rd Paragraph of Article as follows:

In determining the need to investigate fatigue, Table 3.4.1-1 specifies a load factor of ~~1.50~~ 1.75 on the live load force effect resulting from the fatigue truck for the Fatigue I load combination. This factored live load force effect represents the greatest fatigue stress that the bridge will experience during its life.

5.5.3.1 — General

Revise the 5th paragraph of Article as follows:

For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of <u>unfactored</u> effective prestress and permanent loads shall not exceed $0.40f'c$ after losses.

5.5.3.2 — Reinforcing Bars

Revise the 2nd Paragraph of Article as follows:

where:

f_{min} = algebraic minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the unfactored permanent loads or the unfactored permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

5.5.3.4 — Welded or Mechanical Splices of Reinforcement

Revise the 1st Paragraph of Article as follows:

For welded or mechanical connections that are subject to repetitive loads, 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, the constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, shall be as given in Table 5.5.3.4-1.

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5.5.4.2.1 Conventional Construction

Insert the following under the first bullet:

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

Modify the second bullet:

- For tension-controlled precast prestressed concrete sections as defined in Article 5.7.2.1……………………………………...1.00

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C5.5.4.2.1

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

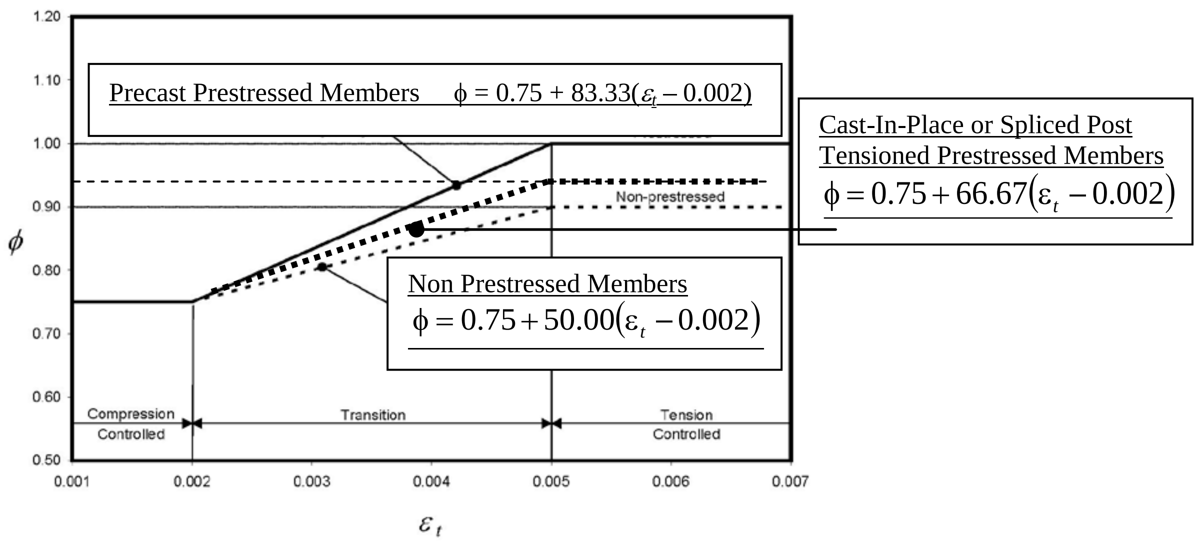

Figure C5.5.4.2.1-1 – Variation of ϕ with Net Tensile Strain ε_t ~~and d_t/c~~ for Grade 60 Reinforcement and for Prestress<u>ed</u>~~ing~~ <u>Members</u>.

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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.

5.6.3.1 — General

Revise the 2nd Paragraph as follows:

The strut-and-tie model ~~should~~ may be considered for the design of deep footings and pile caps or other situations in which the distance between the centers of applied load and the supporting reactions is less than about twice the member thickness.

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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 members with factored axial compressive load less than 0.10 $f'_c A_g$, the net tensile strain in the extreme tension steel at a section shall not be less than 0.004 just as the concrete in compression reaches its assumed strain limit of 0.003.

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. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Sections with a factored axial compressive load that is less than $0.1f'_c A_g$ can be regarded as flexural members. Ensuring that the net tensile strain in the extreme tensile steel is not less than 0.004 is equivalent to the previously established practice of limiting the maximum reinforcement ratio in a cross section to 0.75 times the balanced reinforcement ratio. 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 between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

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5.7.3.3.2 — Minimum Reinforcement

Revise the following definition:

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses)<u>, not including the effects of secondary moment,</u> at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

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5.7.3.4 — Control of Cracking by Distribution of Reinforcement

Revise the 3rd Paragraph as follows:

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance 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.

Add a new paragraph after the 3rd Paragraph:

<u>Class 2 exposure condition applies to all bridge decks. The clear concrete cover to the top reinforcement shall be taken as 2-1/2 in. to determine d_c for use in Eq. 5.7.3.4-1 when verifying reinforcement spacing in bridge decks.</u>

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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.

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.

5.7.3.6.2 — Deflection and Camber

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: 3.0-1.2(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:

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

where:

A'_s = area of compression reinforcement (in^2)

A_s = area of nonprestressed tension reinforcement (in^2)

C5.7.3.6.2

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 4th Paragraph as follows:

The equivalent factored shear force for combined shear and torsion, V_{uT}, shall satisfy:

$$V_{uT} \leq \phi V_n \tag{5.8.2.1-5a}$$

The equivalent factored shear force, ~~V_u,~~ V_{uT}~~,~~ shall be taken equal to:

For solid sections:

$$V_{uT} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \tag{5.8.2.1-6}$$

For the individual web/girder of a box section~~s~~, the combined shear and torsion force is taken from analysis methods defined in Articles 4.6.2, 4.6.3, or:

$$\cancel{V_u + \frac{T_u d_s}{2A_o}} \qquad V_{uT} = V_{ui} + \frac{T_u d_s}{2A_o} \tag{5.8.2.1-7}$$

And

the cross-sectional dimension of the girder shall satisfy the following:

$$\left(\frac{V_u}{b_v d_v}\right) + \left(\frac{T_u}{2A_o b_e}\right) \leq 0.474\sqrt{f_c'} \tag{5.8.2.1-8}$$

where:

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

T_u = factored torsional moment applied to the entire box section (kip-in.)

b_e = effective width of the shear flow path, but not exceeding the minimum thickness of the webs or flanges comprising the closed box section (in). b_e shall be adjusted to account for the presence of ducts.

V_{ui} = factored shear force in the controlling web/girder of the box section (kip)

V_{uT} = equivalent factored shear force from combined shear and torsional effects acting on the individual solid section or equivalent factored shear force from combined shear and torsional effects acting on the controlling web/girder of the box section (kip)

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 the opposite exterior web. In the controlling web, the second term in Eq. 5.8.2.1-7 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 web height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal to that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal.

Add 8th Paragraph as follows:

For cross-section ‘d’ in Table 4.6.2.2.1-1 and segmental box girders, with skewed supports, proper torsion investigation does account for the additional shear generated due to skew support in lieu of applying skew factors from Articles 4.6.2.2.2e, 4.6.2.2.3c or 4.6.2.2.6.

Add 9th Paragraph as follows:

Eq. 5.8.2.1-8 is used to check the cross section dimensions to prevent concrete crushing before yielding of steel stirrups

5.8.2.4 — Regions Requiring Transverse Reinforcement

Add the following to the end of the Article 5.8.2.4:

For footing design, transverse reinforcement is required when V_u exceeds ϕV_c.

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5.8.2.7 — Maximum Spacing of Transverse Reinforcement

Revise the 1st bullet as follows:

- If $v_u < 0.125 f'_c$, then:
  $s_{max} = 0.8 d_v \leq$ ~~24.0 in.~~ 18 in. (5.8.2.7-1)

C5.8.2.7

Add a 2nd Paragraph as follows:

The maximum spacing of the girder shear reinforcement that extends into a cast-in-place concrete deck should be limited to 18 in. based on the recommendations in the report "I-40 Bridge Investigation Final Report" prepared by Wiss, Janney, Elstner Associates, Inc. in Nov 26, 2007.

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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 up to a maximum of 2 in. or one-quarter the diameter of grouted ducts up to a maximum of 1 in. at that level shall be subtracted from the web width for spliced precast girders. 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

Revise the 1st Paragraph as follows:

~~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:

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

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5.8.3.4 — Procedures for Determining Shear Resistance

Revise the 1st Paragraph as follows:

Design for shear may utilize ~~any~~either of the ~~three~~two methods identified herein provided that all requirements for usage of the chosen method are satisfied.

C5.8.3.4

Revise the 1st Paragraph as follows:

~~Three~~Two complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.8.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in Article 5.8.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load.~~ Two approaches are presented in Method 2: a direct calculation, specified in Article 5.8.3.4.2, and an evaluation using tabularized values presented in Appendix B5. The approaches to Method 2 may be considered statistically equivalent. Method 3, specified in Article 5.8.3.4.3, is applicable for both prestressed and nonprestressed sections in which there is no net axial tensile load and at least minimum shear reinforcement is provided. Axial load effects can otherwise be accounted for through adjustments to the level of effective precompression stress, f_{pe}. In regions of overlapping applicability between the latter two methods, Method 3 will generally lead to somewhat more shear reinforcement being required, particularly in areas of negative moment and near points of contraflexure. Method 3 provides a direct capacity rating while Method 2 may require iterative evaluation. If Method 3 leads to an unsatisfactory rating, it is permissible to use Method 2.~~

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5.8.3.4.2 — General Procedure

Delete the entire Article and revise as follows:

The general procedure for determining shear resistance of all prestressed and nonprestressed sections, as described in the provisions of Appendix B5, shall be used.

C5.8.3.4.2

Delete the entire Article and revise as follows:

The general procedure for determining shear resistance of all prestressed and nonprestressed sections, as described in the provisions of Appendix B5, shall be used.

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5.8.3.4.3 — Simplified Procedure for Prestressed and Nonprestressed Sections

Delete entire Article 5.8.3.4.3 and replace with the following:

<u>Article 5.8.3.4.3 "Simplified Procedure for Prestressed and Nonprestressed Sections" shall not be used.</u>

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C5.8.3.5

Add a new 1st Paragraph as follows:

Conservatively, non-concurrent values for M_u and V_u may be used to evaluate longitudinal reinforcement. When coincident values are used, both maximum M_u with coincident V_u, and maximum V_u with coincident M_u, should be checked. If approximate methods are used for the distribution of live loads, the girder distribution factor for bending should be used for both maximum M_{LL} and coincident M_{LL}, and the girder distribution factor for shear should be used for both maximum V_{LL} and coincident V_{LL}. For Strength I, force effects due to both the typical and contraflexure truck configurations should be evaluated

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Add a new article:

C5.8.3.6.2

For cross-section 'd' Table 4.6.2.2.1-1 and segmental box girders, A_ℓ, as defined in Eq. 5.8.3.6.2-1, is used to determine the portions of transverse reinforcement that needs to be closed hoops or 135-degree hooks.

C5.8.3.6.3

Add the following as the 2nd Paragraph:

A_ℓ is distributed around the perimeter of the closed transverse torsion reinforment.

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5.8.4.2 — Computation of the Factored Interface Shear Force, V_{ui}, for Girder/Slab Bridge

Revise the Last Paragraph as follows:

For beam and girders, the longitudinal spacing of the rows of interface shear transfer reinforcing bars shall not exceed ~~24.0~~ 18.0 inch.

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5.8.6 — Shear and Torsion for Segmental Bridges

Delete all the provisions in Article 5.8.6 Shear and Torsion for Segmental Box Girder Bridges.

Add a new Paragraph to Article 5.8.6 Shear and Torsion for Segmental Box Girder Bridges as follows:

Articles 5.8.1, 5.8.2, 5.8.3, 5.8.4, and 5.8.5 shall be used for shear and torsion design of segmental post-tensioned box girders bridges.

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Table 5.9.3-1 Stress Limitations for Prestressing Tendons

Revise Table 5.9.3-1 as follows:

Condition	Tendon Type		
	Stress-Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Prior to Seating: short-term f_{pbt} may be allowed	$0.90f_{py}$	$0.90\ f_{py}$	$0.90f_{py}$
Immediately prior to transfer (f_{pbt})	$0.70\ f_{pu}$	$0.75\ f_{pu}$	---
At service limit state after all losses(f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-tensioning			
~~Prior to Seating short-term f_{pbt} may be allowed~~	~~$0.90\ f_{py}$~~	~~$0.90\ f_{py}$~~	~~$0.90\ f_{py}$~~
Maximum Jacking Stress: short-term f_{pbt} may be allowed	$0.75\ f_{pu}$	$0.75\ f_{pu}$	$0.75\ f_{pu}$
At anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.70\ f_{pu}$	$0.70\ f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.74\ f_{pu}$	$0.70\ f_{pu}$
At service limit state after losses(f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

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

Revise Table 5.9.4.2.2-1 as follows:

Bridge Type	Location	Stress Limit
<u>Segmental and Non-Segmental Bridges</u>	<u>Precompresssed Tensile Zone Bridges, Assuming Uncracked Sections—components with bonded prestressing tendons or reinforcement, subjected to permanent loads only.</u>	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, <u>and/or are located in Caltrans' Environmental Areas I or II.</u> • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, <u>and/or are located in Caltrans' Environmental Area III.</u> • For components with unbonded prestressing tendons.	$0.19\sqrt{f'_c}$ (ksi) $0.0948\sqrt{f'_c}$ (ksi) No tension
Segmentally Constructed Bridges	(*no changes*)	(*no changes*)

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5.9.5.2.2b — Post-tensioned Construction

Revise Table 5.9.5.2.2b-1 as follows:

Type of Steel	Type of Duct	*K* (1/ft)	μ
Wire or strand	Rigid and semi-rigid galvanized metal sheathing	~~0.0002~~	~~0.15-0.25~~
	Tendon Length:		
	< 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
High-strength bars	Galvanized metal sheathing	0.0002	0.30

C5.9.5.2.2b

Add a new last Paragraph as follows:

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

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5.9.5.2.3b — Post-Tensioned Members

Delete Equation 5.9.5.2.3b-1 and replace with the following:

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

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

C5.9.5.2.3b

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

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

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

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5.9.5.3 — Approximate Estimate of Time-Dependent Losses

Add a new last paragraph:

For cast-in-place post-tensioned members, the approximate estimate of time-dependent losses may be taken as a lump sum value of 20 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 pretensioned members and should not be used for post-tensioned structures. Research performed by the University of CA, San Diego (SSRP-11/02) indicates time-dependent losses for cast-in-place post-tensioned box girder bridges are lower than previously expected. A parametric study by Caltrans using equations presented in the aforementioned research indicates losses may range from 11 ksi to 21 ksi. The variance is due to several parameters, such as relative humidity, area of non-prestressing steel and strength of concrete.

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C5.10.4.3.1b

Revise the Paragraph as follows:

A generic ~~stirrup and~~ duct tie detail is shown in Figure C5.10.4.3.1b-1. Small diameter reinforcing bars should be used for better development of these bars. There have been no reported web failures when this detail has used.

Replace Figure C5.10.4.3.1b-1 as follows:

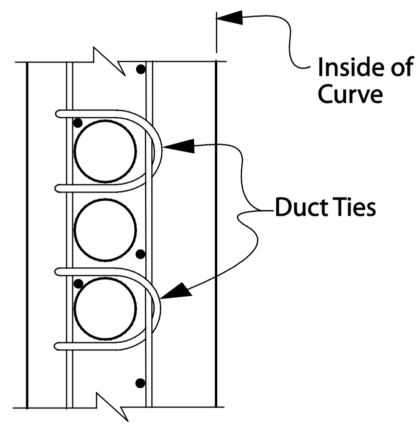

Figure C5.10.4.3.1b-1-Typical ~~Stirrup and~~ Duct Tie Detail

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5.10.5 — External Tendon Supports

Add the following to the end of the Article 5.10.5:

External tendon supports in curved concrete box girders shall be located far enough away from the web to prevent the free length of tendon from bearing on the web at locations away from the supports. When deviation saddles are required for this purpose, they shall be designed in accordance with Article 5.10.9.3.7.

C5.10.5

Add the following:

Deviation saddles in tightly curved bridges may be considered as continuous across the soffit as recommended by Beaupre et. al. (1988).

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5.11.4.3 — Partially Debonded Strands

Revise the 2nd and 3rd Paragraphs as follows:

The number of partially debonded strands ~~should~~ shall 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.

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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 greater than or equal to 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than or equal to 2,000 ppm and/or a pH of less than or equal to 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 at the MHHW elevation.

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.

Delete Table 5.12.3-1 and replace with the following:

Table 5.12.3-1 Minimum Concrete Cover to Reinforcement (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
			Chloride Concentration (ppm)					Chloride concentration (ppm)			
			500-5,000	5,001-10,000	Greater than 10,000			500-5,000	5,001-10,000	Greater than 10,000	
		(a)	(a)	(a)	(a)	(a)	(a), (b)	(a),(b)	(a),(b)	(a),(b)	(a), (c),(e)
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"/ "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 Stirrups, ties and spirals: 1.0 inch										

General Notes:

1. Supplementary cementitious materials (SCM) are required for all exposure conditions period.
2. For protection of bundled bars, ducts and /or prestressing steel, 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 members.
4. For rebar cover in CIDH piles, also refer to Table 10.8.1.3-1.

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) SCMs will be required for enhanced corrosion protection.
(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 shown in the table may be reduced by ½ inch maximum (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.

Add this new article:

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.

Add this new article:

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

Add this new article:

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).

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5.12.4 — Protective Coatings

Delete and replace with the following:

5.12.4 Protection of Exposed Metals

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 for Prestressing Tendons

Delete and replace with the following:

5.12.5 Protection of Concrete Exposed to Acids and Sulfate

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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5.13.4.5.2 — Reinforcing Steel

Revise the 3rd paragraph of Article 5.13.4.5.2 as follows:

For cast-in-place concrete piling, clear distance between parallel longitudinal, and parallel transverse reinforcing bars shall not be less than ~~five times the maximum aggregate size or~~ 5 in~~, except as noted in Article 5.13.4.6 for seismic requirements~~. Radial bundling of longitudinal reinforcement is not allowed in drilled shafts.

Add following paragraph to the end of Article 5.13.4.5.2:

Minimum shear reinforcement in drilled shafts shall be No.5 hoops at 12 in. center to center spacing or equivalent spiral reinforcement, when permitted. Furthermore, if $V_u \geq \varphi V_c$, the requirements of Article 5.8.2.5 shall also apply.

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5.14.1.4.1 — General

Modify the 1st Paragraph as follows:

The provisions of this Article shall apply at the service and strength limit states as applicable. Article 5.14.1.4 need not be applied to design of multi-span bridges composed of precast girders with continuity diaphragms at bent caps.

C5.14.1.4.1

Add a new 1st Paragraph as follows:

Article 5.14.1.4 provides design requirements mainly for multi-span bridges composed of precast girders made continuous with a drop cap bent system. The research to develop these requirements did not consider the inverted T-cap bent system. Caltrans provides a continuous diaphragm by threading dowels through the dapped end of the precast girders prior to pouring concrete between the girder ends. Shear stirrups extend up into the deck, with especially close stirrup spacing at the girder ends. Positive moment is presumed to be transferred into the bent cap. Currently, Caltrans is sponsoring a research study to study and develop connection details between the inverted T-cap bent and precast girders. The requirements of Article 5.14.1.4 may be used as the guidance for design of multi-span bridges composed of precast girders made continuous with the drop cap bent system.

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B5.1 — BACKGROUND

Revise the 1st Paragraph as follows:

~~The general procedure herein is an acceptable alternative to the procedure specified in Article 5.8.3.4.2. The procedure in this Appendix utilizes tabularized values of β and θ instead of Eqs. 5.8.3.4.2-1, 5.8.3.4.2-2, and 5.8.3.4.2-3.~~ Appendix B5 is a complete presentation of the general procedures in LFRD Design (2007)~~ without any interim changes~~.

B5.2— Sectional Design Model General Procedure

Add the following after the 5th Paragraph:

When combined shear and torsion effects must be considered on sections, V_{uT} , as defined in the California Amendment to Article 5.8.2.1, shall be used instead of V_u.

CB5.2 Sectional Design Model General Procedure

Add the following after the 8th Paragraph:

In the calculation of ε_t and ε_x, M_u and V_u may be applied in either of the following combinations:

1. Non-concurrent maximum values for M_u and V_u. This is the more conservative combination.
2. Both of these combinations;
   - Maximum M_u with concurrent V_u, and
   - Maximum V_u with concurrent M_u

If approximate methods, described in Article 4.6.2, are used for the calculation of M_u and V_u, the live load distribution factors shall be applied as follows:

- The live load distribution factors for moment shall be applied to maximum M_{LL} and M_{LL} concurrent with maximum V_{LL}.
- The live load distribution factors for shear shall be applied to maximum V_{LL} and V_{LL} concurrent with maximum M_{LL}.