ADDENDA / ERRATA

To Bridge Design Specifications Users:

Recently, we were made aware of some revisions that need to be applied to the California Amendments to the AASHTO LRFD Bridge Design Specifications – 8th Edition. The most recent revisions are shown in **bold** in the table below. Please replace the previous version of the California Amendments with the version appended herein, which incorporates all errata and addenda to date.

Table for Summary of revisions

CA Amendment page	Article	Date of Revision	Addendum (or) Erratum
3-19A	C3.5.1	October 2022	Addendum
3-31B	3.6.1.8.1 item b	October 2019	Erratum
3-40A	Table 3.7.5-1	September 2019	Erratum
3-40A	Table 3.7.5-1	September 2019	Erratum
3-148A	Eq-3.12.2.3-1	September 2019	Addendum
3-148A	Eq-3.12.2.3-1	September 2019	Erratum
4-38A	Table 4.6.2.2.2b-2	September 2019	Erratum
4-43A	Table 4.6.2.2.3a-2	September 2019	Erratum
4-43A	Table 4.6.2.2.3a-2	September 2019	Erratum
4-46A	Table 4.6.2.2.3c-1	September 2019	Erratum
4-46A	Table 4.6.2.2.3c-1	September 2019	Erratum
4-46A	Table 4.6.2.2.3c-1	September 2019	Erratum
5-34A	5.6.2.1	April 2019	Erratum
5-125B	5.9.2.3.3	May 2022	Addendum
5-125B	C5.9.2.3.3	May 2022	Addendum
5-127A	Eq-5.9.2.3.3-4	May 2022	Addendum
5-127A	Eq-C5.9.2.3.3-1	May 2022	Addendum
5-164C	Table 5.10.1-1	May 2022	Erratum
5-164C	Table 5.10.1-1	May 2022	Addendum
5-164D	Table 5.10.1-1 notes	May 2022	Erratum
5-164D	Table 5.10.1-1 notes	May 2022	Erratum
5-164D	Table 5.10.1-1 notes	May 2022	Addendum
6-52A	Table 6.6.1.2.5-2	October 2022	Erratum
6-160A	6.10.8.2.3	October 2022	Addendum
6-160A	C6.10.8.2.3	October 2022	Addendum
10-126E	C10.8.1.3	June 2021	Addendum
11-111A	A11.3.1	March 2022	Erratum
12-22A	Table 12.6.6.3-1	September 2019	Erratum
13-25A	**A13.4.1**	**December 2023**	**Addendum**
13-25A-B	**A13.4.2**	**December 2023**	**Addendum**

CA Amendment page	Article	Date of Revision	Addendum (or) Erratum
13-25A-B	**CA13.4.2**	**December 2023**	**Addendum**
13-26A	**A13.4.3**	**December 2023**	**Addendum**
14-59B	C14.7.5.3.2	October 2022	Addendum

Page	Existing Text	Revised Text
Section 3		
3-19A		Add C3.5.1 The 10 percent increase in dead load accounts for the weight of the stay-in-place metal form (SIPMF) and additional concrete in the form corrugations.
3-31B	Last sentence in item b) reads: “Live loads shall be placed in the controlling of one or two separate lines chosen to create the most severe conditions.”	Replace with: “Live loads shall be placed in the controlling of one or two separate lanes chosen to create the most severe conditions.”
3-40A	The 4th row in Table 3.7.5-1 column 5 reads: “50%”	Change: “50%” to “100%”
3-40A	The last row in Table 3.7.5-1 column 5 reads: “50%”	Change: “50%” to “0%”
3-148A	Equation 3.12.2.1-1 reads: $\Delta_T = \pm \frac{\alpha L\left(T_{MaxDesign} - T_{MinDesign}\right)}{2}$	Revise equation to read: $\Delta_T = \alpha L\left(T_{MaxDesign} - T_{MinDesign}\right)$
3-148A	Equation number reads: “3.12.2.1-1”	Change equation number: “3.12.2.1-1” to “3.12.2.3-1”
Section 4		
4-38A	8th row in Table 4.6.2.2.2b-2, last column, Range of Applicability, reads: 20 ≤ W <24	Change: “20 ≤ W <24” to 20 ≤ W ≤24
4-43A	The formula in the last row of column 3 in Table 4.6.2.2.3a-2 reads: $3\left(\frac{S}{4.8}\right)^{0.4}\left(\frac{d}{12L}\right)^{0.06}$	Revise formula to read: $3\left(\frac{S}{4.8}\right)^{0.5}\left(\frac{d}{12L}\right)^{0.09}$
4-43A	4th and 8th row in Table 4.6.2.2.3A-2, last column, Range of Applicability, reads: “6 ≤ S <14”	Change: “6 ≤ S <14” to “6 ≤ S ≤14”
4-46A	Several of the last column, Range of Applicability, in Table 4.6.2.2.3c-1 read: “<” signs.	Replace : “<” with “≤”
4-46A	17th row, last column, Range of Applicability, in Table 4.6.2.2.3c-1 reads: “17 < d <110”	Replace “17 < d <110” to “17 ≤ d ≤60”
4-46A	4th, 9th, and 14th row, last column, Range of Applicability, in Table 4.6.2.2.3c-1 read: “>” signs	Replace “>” with “≥”

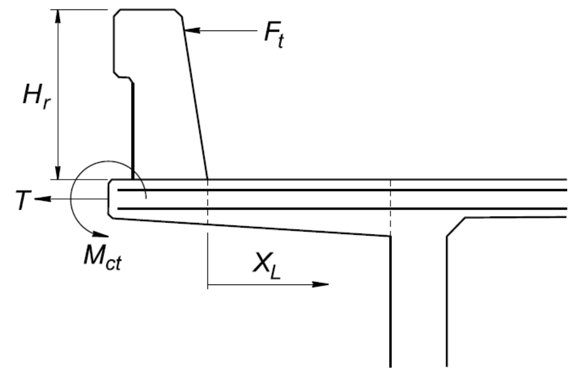
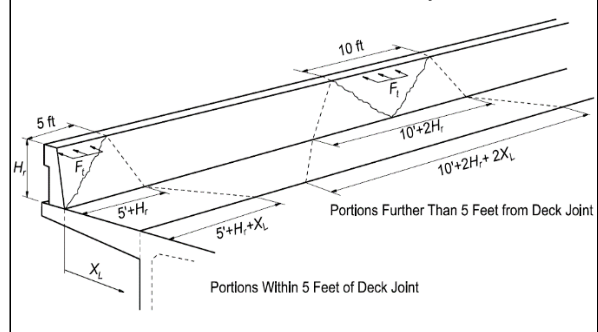
Page	Existing Text	Revised Text
Section 5		
5-34A	Amendment to the 11th bullet reads: "Sections are tension-controlled where the net tensile strain in the extreme tension steel is equal to or greater than the tension-controlled strain limit, ε_{tl} just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and the tension-controlled strain limit constitute a transition region between compression controlled and tension-controlled sections. For nonprestressed concrete members with factored axial compressive load less than $0.1f'_cA_g$, 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. The tension-controlled strain limit, ε_{tl} , shall be taken as 0.005 for nonprestressed reinforcement with a specified minimum yield strength, f_y = 100 ksi. For nonprestressed reinforcement with a specified minimum yield strength between 75.0 and 100 ksi, the tension-controlled strain limit shall be determined by linear interpolation based on specified minimum yield strength."	Revise to: "Sections are tension-controlled where the net tensile strain in the extreme tension steel is equal to or greater than the tension-controlled strain limit, ε_{tl}, 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 the tension-controlled strain limit constitute a transition region between compression-controlled and tension-controlled sections. For nonprestressed concrete members with factored axial compressive load less than $0.1f'_cA_g$, 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. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.005 for nonprestressed reinforcement with a specified minimum yield strength, $f_y \leq 75$ ksi and prestressed reinforcement. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.008 for nonprestressed reinforcement with a specified minimum yield strength, f_y = 100 ksi. For nonprestressed reinforcement with a specified minimum yield strength between 75.0 and 100 ksi, the tension-controlled strain limit shall be determined by linear interpolation based on specified minimum yield strength."
5-125B		Add 1st paragraph of 5.9.2.3.3—Principal Tensile Stresses in Webs as follows: "Except for nonsegmental cast-in-place prestressed concrete box girders with conventional geometries, the provisions specified herein shall apply to all types of post-tensioned superstructures with internal and/or external tendons. The provisions specified herein shall also apply to pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. As maximum principal tensions may not occur at the neutral axis, various locations along the height of the web shall be checked."

Page	Existing Text	Revised Text
Section 5		
5-125B		Add 1st paragraph of C5.9.2.3.3 as follows: "The principal stress check is introduced to limit web cracking at the service limit state for all types of post-tensioned superstructures with internal and/or external tendons and pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. Experience has shown that the cracking in the webs of conventional pretensioned girders with a compressive strength of concrete for use in design up to 10.0 ksi and in the webs of nonsegmental cast-in-place prestressed concrete box girders with conventional geometries has not been a problem. In the context of this article, nonsegmental cast-in-place prestressed concrete box girders with unconventional geometries include, but are not limited to, any of the following: • Variable structure depths • Structure depths greater than 12 feet • Span lengths greater than 300 feet • Minimum depth-to-span ratios less than 0.045 and 0.040 for simple and continuous spans respectively for vehicular bridges • Girder spacing to depth ratios greater than 2.0 • Individual web thicknesses less than 12 inches • Webs with openings larger than 10% of the structure depth • Any superstructure where a single spine beam analysis is not allowed"
5-127A		Add revised Equation 5.9.2.3.3-4 as follows: $f_{\min} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) - \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$ (5.9.2.3.3-4)
5-127A		Add revised Equation C5.9.2.3.3-1 as follows: $f_{\max} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) + \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$ (C5.9.2.3.3-1)

Page	Existing Text	Revised Text
Section 5		
5-164C	8th row in Table 5.10.1-1, first column reads: “Exposed faces of box girder webs and all other exposed girders. Bent caps, diaphragms and hinged joints(f)”	Replace with: “Exposed faces of box girder webs and all other exposed girders, bent caps, diaphragms, and hinged joints(f)”
5-164C	Table 5.10.1-1	Add three rows to Table 5.10.1-1
5-164D	General Notes 1 reads: “Except for the Non-corrosive Environment, all exposure conditions must meet the Supplementary Cementitious Materials (SCM) requirements of Section 90, “Concrete in Corrosive Environments” that correspond to the specific environment.”	Replace with: “Except for the Non-corrosive Environment, all exposure conditions must meet the Supplementary Cementitious Materials (SCM) requirements of Caltrans Standard Specifications Section 90, “Concrete in Corrosive Environments” that correspond to the specific environment.”
5-164D	General Notes 2 reads: “For protection of bundled bars, ducts, and/or pre-stressing steel, see Articles 5.10.1.	Replace with: “For protection of bundled bars, ducts, and/or pre-stressing steel, see Article 5.10.1.”
5-164D	General Notes 5 reads: “For concrete surfaces not exposed to weather, soil, or water, the minimum concrete cover to principal reinforcement is 1.5 inches and to stirrups, ties, and spirals is 1.0 inch.”	Delete General Notes 5
Section 6		
6-52A	Table 6.6.1.2.5-2 row 3, column 2 reads: 1.5 (HL-93)	Move 1.5 (HL-93) to row 4 column 2.
6-160A	Bullet 3 under 6.10.8.2.3 reads: • For revise curvature bending……	Delete Bullet 3
6-160B		Add a bullet under C6.10.8.2.3

Page	Existing Text	Revised Text
Section 10		
10-126E	First and second paragraphs read: “In order to improve concrete flow when constructing drilled shafts, a 5 in. x 5 in. clear window between the horizontal and vertical shaft reinforcing steel shall be maintained, except at the locations of the inspection pipes where the minimum clear spacing between the longitudinal reinforcing bars and the inspection pipes is 3.0 in. The maximum center-to-center spacing of longitudinal bars in drilled shafts (CIDH Piles) is limited to 10 in. when the shaft diameter is less than 5 ft., and 12 in. for larger shafts, except at the locations of inspection pipes where 8.5 in of clear spacing between the main longitudinal bars is required.”	Replace with, “In order to improve concrete flow when constructing drilled shafts, a minimum 5 in. x 5 in. clear window between the horizontal and vertical shaft reinforcing steel shall be maintained. The maximum center-to-center spacing of longitudinal bars in drilled shafts (CIDH Piles) is limited to 10 in. when the shaft diameter is less than 5 ft., and 12 in. for larger shafts. At locations of inspection pipes, 8.5 in. of clear spacing shall be provided between the main longitudinal reinforcing bars adjacent to the inspection pipe, and the minimum clear spacing between the longitudinal reinforcing bars and the inspection pipe shall be 3.0 in.”
Section 11		
11-111A	Equation (A11.3.1-2) reads: $\delta = \tan^{-1}\left\{\frac{\sin(2\theta_{Mo}) + m_\alpha \sin(2i)}{2\left[\sin^2\theta_{Mp} - m_\alpha \cos^2 i\right]}\right\}$ (degrees)	Revise Equation (A11.3.1-2) as follows: $\delta = \tan^{-1}\left\{\frac{\sin(2\theta_{Mo}) + m_\alpha \sin(2i)}{2\left[\sin^2\theta_{Mp} + m_\alpha \cos^2 i\right]}\right\}$ (degrees)
Section 12		
12-22A	Several “greater than” signs in the last column, Minimum Cover, in Table 12.6.6.3-1	Change “>” signs to “≥” signs
Section 13		
13-25A		Add a paragraph after the last paragraph of A13.4.1 as follows: Design Case 1 and Design Case 2 are also applicable for the evaluation of existing deck overhangs supporting new railings.
13-25A	The title of A13.4.2 reads: **A13.4.2—Decks Supporting Concrete Parapet Railings**	Replace with: **A13.4.2—Decks Supporting Solid Concrete Parapet Railings and Post-and-Beam Railings with Continuous Concrete Curbs**

Page	Existing Text	Revised Text
Section 13		
13-25A	The first paragraph of A13.4.2 reads: For Design Case 1, the deck overhang may be designed to provide a flexural resistance, *Ms*, in kip-ft/ft which, acting coincident with the tensile force *T* in kip/ft, specified herein, exceeds *Mc* of the parapet at its base. The axial tensile force, *T*, may be taken as $T = \frac{R_w}{L_c + 2H}$ (A13.4.2-1) where: *Rw* = parapet resistance specified in Article A13.3.1 (kips) *Lc* = critical length of yield line failure pattern (ft) *H* = height of wall (ft) *T* = tensile force per unit of deck length (kip/ft)	Replace with: For Design Case 1, the load *CT* applied to the deck overhang from the vehicular collision force on the railing shall be the combined force effects of transverse tensile force *T* and transverse moment M_{ct} as follows: For portions of the overhang located further than 5 feet from a deck joint: $T = \frac{F_t}{10 + 2H_r + 2X_l}$ (A13.4.2-1) $M_{ct} = \frac{F_t H_r}{10 + 2H_r + 2X_L}$ (A13.4.2-2) For portions of the overhang located within 5 feet of a deck joint: $T = \frac{F_t}{5 + H_r + X_L}$ (A13.4.2-3) $M_{ct} = \frac{F_t H_r}{5 + H_r + X_L}$ (A13.4.2-4) where: F_t = transverse traffic railing design force from Table A13.2-1 (kips) H_r = height of railing (ft) M_{ct} = deck overhang transverse moment per unit length due to F_t (kip-ft/ft) *T* = deck overhang transverse tensile force per unit length due to F_t (kip/ft) X_L = transverse distance from the toe of railing to the deck overhang section being considered (ft) The effects on the deck overhang of the longitudinal design force, F_L, from Table A13.2-1 and the effects of deck punching shear from the railing collision forces may be ignored. The flexural resistance of the deck overhang shall be determined in accordance with Section 5, with the following additional requirements for deck overhangs on existing bridges: • Both top and bottom transverse deck reinforcement shall be considered. • The expected yield strength of the existing deck overhang reinforcement and the expected concrete compressive strength of the deck overhang shall be used in lieu of the specified minimum

Page	Existing Text	Revised Text
Section 13		
13-25A Cont.		yield strength and specified concrete compressive strength, respectively.  **Figure A13.4.2-1 – Force Effects on Deck Overhang Due to Railing Collision Force**
13-25A	CA13.4.2 reads: If the deck overhang capacity is less than that specified, the yield line failure mechanism for the parapet may not develop as shown in Figure CA13.3.1-1, and Eqs. A13.3.1-1 and A13.3.1-2 will not be correct. The crash testing program is oriented toward survival, not necessarily the identification of the ultimate strength of the railing system. This could produce a railing system that is significantly overdesigned, leading to the possibility that the deck overhang is also overdesigned.	Delete the entire commentary CA13.4.2 and replace with the following: Concrete Barrier Type 836 and Concrete Barrier Type 842 are examples of solid concrete parapet railings. Concrete Barrier Type 85 and California ST-75 Bridge Rail are examples of post-and-beam railings with continuous concrete curbs. The effective lengths of the deck overhang used to resist the railing collision force are illustrated in Figure CA13.4.2-1. The effective lengths were determined based on the results of static nonlinear finite element analyses.  **Figure CA13.4.2-1 – Effective Deck Overhang Lengths Resisting Railing Collision Force** For the expected yield strength of the existing deck overhang reinforcement and the expected concrete compressive strength of the deck overhang, see Bridge Design Memo 16.4.

Page	Existing Text	Revised Text
Section 13		
13-26A	The title of A13.4.3 reads: **A13.4.2—Decks Supporting Post-and-Beam Railings**	Replace with: **A13.4.3—Decks Supporting Post-and-Beam Railings Without Continuous Concrete Curbs**
Section 14		
14-59B	C14.7.5.3.2 Last sentence reads: Additionally, if the bearing is originally set or reset at the average of the design thermal movement range computed in accordance with Article 3.12.2 may be substituted for 75 percent as specified.	Replace with: Additionally, if the bearing is originally set or reset at the average of the design temperature range, 50 percent of the design thermal movement range computed in accordance with Article 3.12.2 may be substituted for 75 percent as specified.

California Amendments
to the AASHTO LRFD Bridge Design Specifications
(2017 Eighth Edition)

April 2019

DEPARTMENT OF TRANSPORTATION
STATE OF CALIFORNIA

Foreword

In 1993, the AASHTO Subcommittee on Bridge and Highway Structures (SCOBS) voted to accept the *AASHTO LRFD Bridge Design Specifications* as an alternate design specification. In June 2000, FHWA mandated that LRFD be used on all new bridge design commencing on or after October 1, 2007 and provided additional information in a clarification memorandum dated January 22, 2007.

In 1999, California Department of Transportation (Caltrans) began developing amendments to the *AASHTO LRFD Bridge Design Specifications* that were necessary to adopt the national code into California's bridge design practice. In December 2004, Richard D. Land, former State Bridge Engineer, established April 2006 as the transition date to use the LRFD specifications for bridges designed by the State. Similarly, October 2006 was established for using the LRFD specifications for bridges designed by Local Agencies or others located within state right-of-way.

In April 2006, Kevin J. Thompson, State Bridge Engineer, confirmed that all structural components for bridges designed by the State that had not received Type Selection approval, shall conform to the *AASHTO LRFD Bridge Design Specifications, 3rd Edition, with 2005 Interim Revisions, as amended by Caltrans.* Similarly, October 1, 2006 was confirmed for the LRFD structural design for bridges, without Type Selection approval, designed by Local Agencies or others located within state right-of-way. Full implementation of the complete the *AASHTO LRFD Design Specifications* including the geotechnical design of foundations was set for April 1, 2007 for bridges designed by the State and October 1, 2007 for bridges designed by others.

In December 2008, Kevin J. Thompson, State Bridge Engineer, approved the *AASHTO LRFD Bridge Design Specifications, 4th Edition with the California Amendments*, as the primary Caltrans bridge design specifications. In September 2010, Tony Marquez, Deputy Division Chief, expanded this requirement to include earth retaining structures. In December 2011, Barton Newton, State Bridge Engineer approved updates to Sections 2, 3, 4, 5, 6, 10, 11, 12, and 13.

In March 2014, Barton Newton, State Bridge Engineer, approved the *AASHTO LRFD Bridge Design Specifications, Sixth Edition with the California Amendments, January 2014* as the primary Caltrans bridge design specifications.

In August 2019, Ruth Fernandes, State Bridge Engineer (A), approved the *AASHTO LRFD Bridge Design Specifications, 8th Edition with the California Amendments* as the primary Caltrans bridge design specifications. The LRFD Specifications with the most current California amendments shall be the basis for all advance planning studies, geotechnical investigation, bridge design and other project supporting documentation and bridge design guidance material.

PREFACE
to
CALIFORNIA AMENDMENTS

CALTRANS STANDARD SPECIFICATIONS (CURRENT VERSION):

Shall supersede all references to the *AASHTO LRFD Bridge* ***Construction*** *Specifications* within the *AASHTO LRFD Bridge Design Specifications*. However, the AASHTO Construction Specifications are recommended as reference.

THE GENERAL PLAN TITLE BLOCK SHALL SPECIFY THE DESIGN LIVE LOAD AS:

"Load and Resistance Factor Design", and "HL93 w/ 'Low-Boy' and Permit Design Vehicle"

THE GENERAL NOTES SHALL SPECIFY:

"Load and Resistance Factor Design" and list the "*AASHTO LRFD Bridge Design Specifications*, 8th edition with California Amendments".

1.2—DEFINITIONS

Replace the definition:

Service Life—The period of time that the bridge is expected to be in operation. The service life for new construction is considered to be 75 years unless otherwise specified.

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1.3—DESIGN PHILOSOPHY

1.3.3—Ductility

Replace the article with the following:

The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength limit state before failure. The structural system of a bridge shall be proportioned and detailed to ensure a significant inelastic deformation capacity at the extreme event limit state to ensure an extremely low probability of collapse.

Energy-dissipating devices may be substituted for or used to supplement conventional ductile earthquake resisting systems and the associated methodology addressed in these Specifications or the *AASHTO Guide Specifications for Seismic Design of Bridges.*

For all limit states:

η_D = 1.00

C1.3.3

Add a new last paragraph as follows:

A value of 1.0 is being used for η_D until its application is better defined.

1.3.4—Redundancy

Replace the article with the following:

Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them.

For all limit states:

η_R = 1.00

C1.3.4

Add a new last paragraph as follows:

A value of 1.0 is being used for η_R until its application is better defined.

1.3.5—Operational Importance

Replace paragraphs 3 and 4 with the following:

For all limit states:

η_I = 1.00

C1.3.5

Add a new last paragraph as follows:

A value of 1.0 is being used for η_I until its application is better defined.

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2.3.2.2.2—Protection of Users

Replace the 3rd paragraph with the following:

In the case of movable bridges, warning signs, lights, signal bells, gates, barriers, and other safety devices shall be provided for the protection of pedestrian, cyclists, and vehicular traffic. These shall be designed to operate before the opening of the movable span and to remain operational until the span has been completely closed. The devices shall conform to the requirements for "Traffic Control for Movable Bridges," in the *California Manual on Uniform Traffic Control Devices* (CA MUTCD) or as shown on plans.

2.3.2.2.3—Geometric Standards

Replace the article with the following:

Requirements of the Caltrans *Highway Design Manual* shall either be satisfied or exceptions thereto shall be justified and documented. Width of shoulders and geometry of traffic barriers shall meet the specifications of the Owner.

2.3.2.2.4—Road Surfaces

Replace the article with the following:

Road surfaces on a bridge shall be given crown, drainage, and superelevation in accordance with the Caltrans *Highway Design Manual* or local requirements.

Add the following commentary:

C2.3.2.2.4

Bridge deck surface characteristics are specified in Caltrans *Standard Specifications.*

2.3.3.2—Highway Vertical

Replace the article with the following:

The vertical clearance of highway structures shall be in conformance with the Caltrans *Highway Design Manual* for the Functional Classification of the Highway or exceptions thereto shall be justified. Possible reduction of vertical clearance, due to settlement of an overpass structure, shall be investigated. If the expected settlement exceeds 1.0 in., it shall be added to the specified clearance.

The vertical clearance to sign supports and pedestrian overpasses shall be in conformance with the Caltrans *Highway Design Manual*.

The vertical clearance from the roadway to the overhead cross bracing of through truss structures should not be less than 17.5 ft.

2.3.3.3—Highway Horizontal

Replace the 2nd paragraph with the following:

Horizontal clearance under a structure should meet the requirements of Article 2.3.2.2.

2.6.4.4.2—Bridge Scour

Replace the 3rd paragraph with the following:

Spread footings on soil or erodible rock shall be located so that the top of footing is below the total scour elevation and the bottom of footing is below the scour depths determined for the check flood for scour. Spread footings on scour-resistant rock shall be designed and constructed to maintain the integrity of the supporting rock.

Replace the 4th paragraph with the following:

Deep foundations with footings shall be designed to place the top of the footing below the estimated degradation plus contraction scour depth where practical to minimize obstruction to flood flows and resulting local scour. Even lower elevations should be considered for pile-supported footings where the piles could be damaged by erosion and corrosion from exposure to stream currents. Where conditions dictate a need to construct the top of a footing to an elevation above the total scour elevation, attention shall be given to the scour potential of the design

C2.6.4.4.2

Add the following after the 3rd paragraph:

Total scour is the cumulative sum of contraction, degradation, and local scour. Figure C2.6.4.4.2-1 shows a typical spread footing foundation.

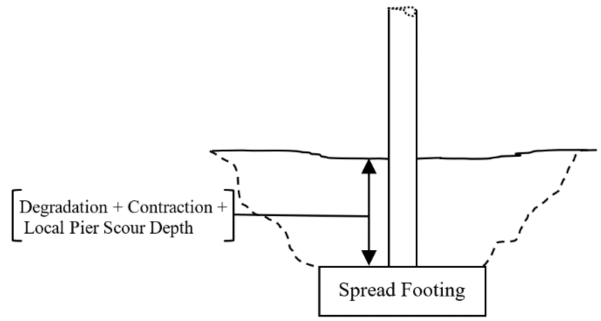

Figure C2.6.4.4.2-1 Spread Footing Location

Add the following after the 4th paragraph:

Foundations should be designed to withstand the conditions of scour. In general, this will result in deep foundations. Figure C2.6.4.4.2-2 shows a typical deep foundation.

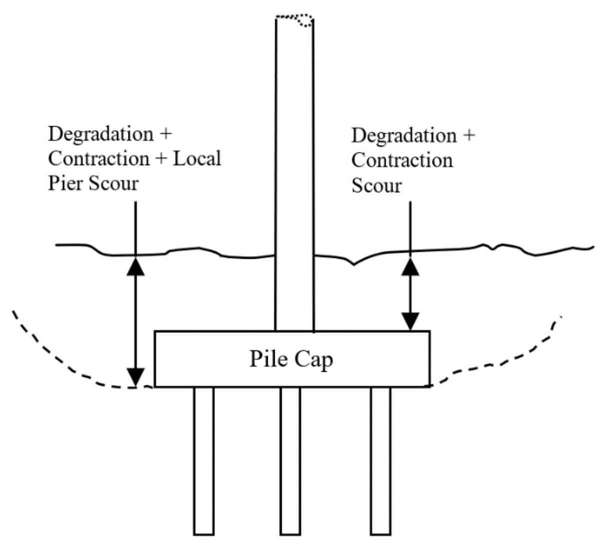

Figure C2.6.4.4.2-2 Deep Foundation Location

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2.6.6.3—Type, Size, and Number of Drains

C2.6.6.3

Replace the 1st paragraph with the following:

For further guidance or design criteria on bridge deck drainage, see the Caltrans *Highway Design Manual, Memo to Designers, and Bridge Design Aids*.

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3.3.2—Load and Load Designation

Add the following notations:

DC_{Sub} = dead load of structural components and nonstructural attachments of substructure
DC_{Sup} = dead load of structural components and nonstructural attachments of superstructure
ES_H = earth surcharge horizontal load
ES_V = earth surcharge vertical load

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3.4.1—Load Factors and Load Combinations

Replace the following notation in the 1st paragraph:

γ_i = load factors specified in Tables 3.4.1-1, 3.4.1-2, 3.4.1-3, 3.4.1-4, 3.4.5.1-1 and 3.4.5.1-2.

Replace the 2nd bullet in the 2nd paragraph with the following:

- Strength II—Load combination relating to the use of the bridge by Owner specified special design vehicles, evaluation permit vehicles, or both without wind. The Caltrans specified special design vehicle and evaluation permit vehicle shall be the Permit Vehicle as specified in Article 3.6.1.8.

C3.4.1

Replace the 2nd paragraph with the following:

The vehicular braking force is not included in this load combination.

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Replace the 6th bullet of the 2nd paragraph of the article with the following:

- Extreme Event I – Load combination including earthquake. The load factor for live load, γ_{EQ}, shall be determined on a project-specific basis for operationally important structures. For standard bridges $\gamma_{EQ} = 0.0$

Replace the 9th paragraph of the commentary with the following:

Vehicular live loads have not been observed to be in-phase with the bridge structure during seismic events. Thus, the inertial effect of actual live loads on typical bridges is assumed to be negligible. Bridges that were seismically retrofitted without consideration of vehicular loads performed well during the 1994 Northridge earthquake.

Replace the 4th bullet of the 10th paragraph of the commentary with the following:

- Although these limit states include water loads, *WA*, the effects due to *WA* are considerably less significant than the effects on the structure stability due to scour. Therefore, unless specific site conditions dictate otherwise, local pier scour depths should not be combined with *BL*, *EQ*, *CT*, *CV*, or, *IC* in the structural or geotechnical design. However, the effects due to degradation and contraction scour of the channel should be considered.

Replace the 5th bullet of the 10th paragraph of the commentary with the following:

- The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure may undergo considerable inelastic deformation by which locked-in-force effects due to *TU*, *TG*, *CR*, *SH* and *SE* are expected to be relieved. The effects due to degradation and contraction scour should be considered for both structural and geotechnical design.

Replace the 13th bullet of the 2nd paragraph of the article with the following:

- Fatigue II—Fatigue and fracture load combination related to finite load-induced fatigue life due to one permit truck (P9) specified in Article 3.6.1.4.1.

Replace the 23rd paragraph of the commentary with the following:

Finite fatigue life is the design concept used for lower traffic volume bridges. The effective fatigue stress range is kept lower than the fatigue resistance, which is a function of load cycles and details, to provide a finite fatigue life. The load factor for the Fatigue II load combination, applied to a single design truck, reflects a load level found to be representative of the permit truck population with respect to a small number of stress range cycles and to their cumulative effects in steel elements, components, and connections for finite fatigue life design.

Add the following after the 2nd paragraph of the article:

Load combinations applicable to abutment construction conditions have been added as cases I and II:

- Construction I - Load combination related to the construction condition where the abutment has been built but the superstructure has not been constructed. For post-tensioned superstructures, when considering Construction I load combination, lateral soil pressure shall be calculated using the height of the abutment below the backwall.
- Construction II- Load combination related to construction condition, where soil surrounding the abutment has been removed for repair, widening, or other reasons after the superstructure has been constructed.

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Replace the 10th paragraph of the article with the following:

The load factor for settlement, γ_{SE}, shall be taken as:

1. For predefined settlements used for geotechnical design of foundations, that is 1.0 in. for continuous spans and simple spans with diaphragm abutments and 2.0 in. for simple spans with seat abutments:
   - When geotechnical information indicates that actual differential settlement is not expected to exceed 0.5 in., settlement does not need to be considered in the design of the superstructure.
   - When geotechnical information indicates that differential settlement is likely to exceed 0.5 in., force effects due to predefined settlements shall be included in the design of the superstructure, and the load factor γ_{SE} shall be taken as 0.5 and 0.0.
2. For refined analysis using nonlinear soil springs, the force effects due to settlement are directly included in the structural analysis. In that case settlement load factor γ_{SE} shall be taken as 1.0 and 0.0.

Replace Table 3.4.1-1 with the following:

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	$LL_{HL\text{-}93}$ *IM CE BR PL LS*	LL_{Permit} *IM CE*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
											EQ	*BL*	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	0	1.00	0	0	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
STRENGTH II	γ_p	0	1.35	1.00	0	0	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
STRENGTH III	γ_p	0	0	1.00	1.00	0	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
STRENGTH IV	γ_p	0	0	1.00	0	0	1.00	0.50/ 1.20	0	0	0	0	0	0	0
STRENGTH V	γ_p	1.35	0	1.00	1.00	1.00	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
EXTREME EVENT I	1.00	γ_{EQ}	0	1.00	0	0	1.00	0	0	0	1.00	0	0	0	0
EXTREME EVENT II	1.00	0.50	0	1.00	0	0	1.00	0	0	0	0	1.00	1.00	1.00	1.00
SERVICE I	1.00	1.00	0	1.00	1.00	1.00	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
SERVICE II	1.00	1.30	0	1.00	0	0	1.00	1.00/ 1.20	0	0	0	0	0	0	0
SERVICE III	1.00	γ_{LL}	0	1.00	0	0	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}	0	0	0	0	0
SERVICE IV	1.00	0	0	1.00	1.00	0	1.00	1.00/ 1.20	0	1.00	0	0	0	0	0
FATIGUE I $LL_{HL\text{-}93}$, *IM* & *CE* only	0	1.75	0	0	0	0	0	0	0	0	0	0	0	0	0
FATIGUE II LL_{Permit} , *IM* & *CE* only	0	0	1.00	0	0	0	0	0	0	0	0	0	0	0	0

Add Article 3.4.5 as follows:

3.4.5—Load Factors for Abutments

Abutments shall be designed for the Service, Strength, Extreme Event, and Construction limit states specified in Articles 3.4.5.1 and 3.4.5.2. The maximum horizontal shear force transferred from the superstructure to a non-integral abutment may be assumed as 20% of the sum of the *DC* and *DW* reactions, that is 0.2(*DC*+*DW*). For this shear force, a load factor of 1.25 shall be used for both *DC* and *DW* for the Strength Limit State combinations.

3.4.5.1—Service, Strength, and Construction Load Combinations

Abutments shall be designed for the Service-I load combination in Table 3.4.1-1 and the Strength, and Construction load combinations specified in Table 3.4.5.1-1. For γ_p values of abutments refer to Table 3.4.5.1-2. For dynamic load allowance (*IM*) of abutments, refer to Article 3.6.2.1.

Table 3.4.5.1-1—Strength and Construction Load Factors for Abutments

Combination	DC_{Sup}	DC_{Sub}	DD	DW	EH, ES_H EV ES_V	LL_{HL93} IM CE BR PL LS	LL_{Permit} IM CE	WA	WS	WL	TU	PS CRS H
Strength I	γ_p	γ_p	γ_p	γ_p	γ_p	1.75	0	1.00	0	0	1.00	1.00
Strength II	γ_p	γ_p	γ_p	γ_p	γ_p	0	1.35	1.00	0	0	1.00	1.00
Strength III	γ_p	γ_p	γ_p	γ_p	γ_p	0	0	1.00	1.00	0	1.00	1.00
Strength V	γ_p	γ_p	γ_p	γ_p	γ_p	1.35	0	1.00	1.00	1.00	1.00	1.00
Construction I	0	γ_p	0	0	γ_p	0	0	0	0	0	0	0
Construction II	1.25	1.25	0	1.50	0	0	0	0	0	0	1.00	1.00

Table 3.4.5.1-2—Load Factors for Permanent Loads, γ_p (for Abutments)

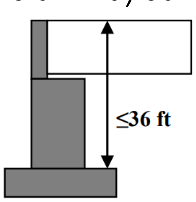
Type of Load and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC_{Sub}: Dead Load of Structure Components and Nonstructural Attachments of Substructure		1.25	0.90
DC_{Sup}: Dead Load of Structure Components and Nonstructural Attachments of Superstructure		1.25	0.90
DD: Downdrag	Pile, α Tomlinson Method	1.40	0.25
	Pile, λ Method	1.05	0.30
	Drilled Shaft, O’Neill and Reese (2010) Method	1.25	0.35
DW: Dead load of Wearing Surface and Utilities		1.50	0.65
EH: Active Horizontal Earth Pressure		1.50	0.75
ES_H: Earth Surcharge Horizontal Load		1.50	0.75
ES_V: Earth Surcharge Vertical Load		1.35	1.00
EV: Vertical Earth Pressure		1.35	1.00

3.4.5.2—Extreme Event-I (Seismic) Load Combination

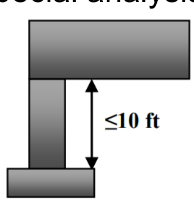
If an abutment in Type S1 (as defined in Article 6.1.2 of SDC version 2.0) soil meets the following height limitations, seismic forces shall be considered **only** in global stability analysis of the abutment:

- The height measured from the superstructure deck to the bottom of the stem is not greater than 36 ft for non-integral abutments.
- The height measured from the superstructure soffit to the bottom of the stem is not greater than 10 ft for integral abutments.

Components of abutments such as shear keys are checked for seismic effects per Caltrans Seismic Design Criteria (SDC). Abutments that do not meet the above limitations and/or are located in Type S2 (as defined in Article 6.1.3 of SDC version 2.0) soil require special analysis.



Non-Integral Type Abutment (with/without piles)



Integral Type Abutment (with/without piles)

3.5.1—Dead Loads: *DC*, *DW*, and *EV*

Add the following after the 2nd paragraph:

The dead load, *DC*, of cast-in-place concrete decks between precast concrete and steel girder flange edges shall be increased by 10 percent.

A future wearing surface load of 35 psf of roadway shall be included in the superstructure dead load, *DW*. This load is in addition to any surface or deck seal provided in the structure.

C3.5.1

Add a new paragraph as follows:

The 10 percent increase in dead load accounts for the weight of the stay-in-place metal form (SIPMF) and additional concrete in the form corrugations.

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3.6.1.2.6a—General

Replace the 2nd and 3rd paragraphs with the following:

Live load shall be distributed to the top slabs of flat top three-sided, box, or long-span concrete arch culverts with less than 2.0 ft of fill as specified in Article 4.6.2.10. For unique situations, such as existing culverts or extensions, round culverts with less than 1.0 ft of fill shall be analyzed with more comprehensive methods such as finite element method considering soil-structure interaction.

Where the depth of fill over round culverts is greater than 1.0 ft, or when the depth of fill over flat top three-sided, box, or long-span concrete arch culverts is 2.0 ft or greater the live load shall be distributed to the top surface of the structure as wheel loads, uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area specified in Article 3.6.1.2.5 increased by the live load distribution factors (LLDF) specified in Table 3.6.1.2.6a-1, and the provisions of Articles 3.6.1.2.6b and 3.6.1.2.6c. More precise methods of analysis may be used.

Replace Table 3.6.1.2.6a-1 with the following:

Table 3.6.1.2.6a-1—Live Load Distribution Factor (LLDF) for Buried Structures

Structure Type	LLDF Transverse or Parallel to Span
Concrete Pipes	1.15 for diameters 2.0 ft or less 1.75 for diameters 8.0 ft or greater Linearly interpolate for LLDF between these limits
All other culverts and buried structures	1.15

3.6.1.2.6b—Traffic Parallel to the Culvert Span

Replace the equation 3.6.1.2.6b-1 with the following:

$$H_{int\text{-}t} = \frac{s_w - \frac{w_t}{12} - \frac{0.06D_i}{12}}{LLDF} \qquad (3.6.1.2.6b\text{-}1)$$

Replace equation 3.6.1.2.6b-6 with the following:

- where $H \geq H_{int\text{-}p}$:

$$l_w = \frac{l_t}{12} + s_a + LLDF(H) \quad (3.6.1.2.6b\text{-}6)$$

3.6.1.3.1—General

Add a 4th bullet to the 1st paragraph as follows:

- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 100 percent of the effect of two design tandems spaced anywhere from 26.0 ft to 40.0 ft from the rear axle of the leading tandem to the lead axle of the other, combined with 100 percent of the design lane load specified in Article 3.6.1.2.4. The two design tandems shall be placed in adjacent spans to produce maximum force effects.

C3.6.1.3.1

Replace the 3rd paragraph with the following:

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on “low boy” type vehicles weighing up to about 110 kip. In California, side-by-side occurrences of the “low boy” truck configuration are routinely found. This amendment is consistent with Article 3.6.1.2.1, will control negative bending serviceability in two-span continuous structures with 20-ft to 60-ft span lengths, and should not be considered a replacement for the Strength II Load Combination.

3.6.1.3.3—Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts.

C3.6.1.3.3

Add a new 5th paragraph as follows:

The force effects due to one 32.0-kip axle on the strip-widths specified in Table 4.6.2.1.3-1, were found to be similar to Caltrans' past practice and envelope two 24.0-kip axles spaced 4'-0" on center (design tandem). Also, the 54.0-kip tandem axle of the permit vehicle typically doesn't control deck designs when applying the appropriate load factors or allowable stresses.

3.6.1.4—Fatigue Load

3.6.1.4.1—Magnitude and Configuration

Replace the 1st paragraph with the following:

For the Fatigue I limit state, the fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 30.0 ft. between the 32.0-kip axles.

Add the following after the 2nd paragraph:

For the Fatigue II limit state, the fatigue load, LLpermit, shall be one permit truck, P9, as specified in Figure 3.6.1.4.1-2.

C3.6.1.4.1

Add the following paragraph:

The permit truck, P9, specified in Figure 3.6.1.4.1-2 represents the majority of permit trucks allowed in California.

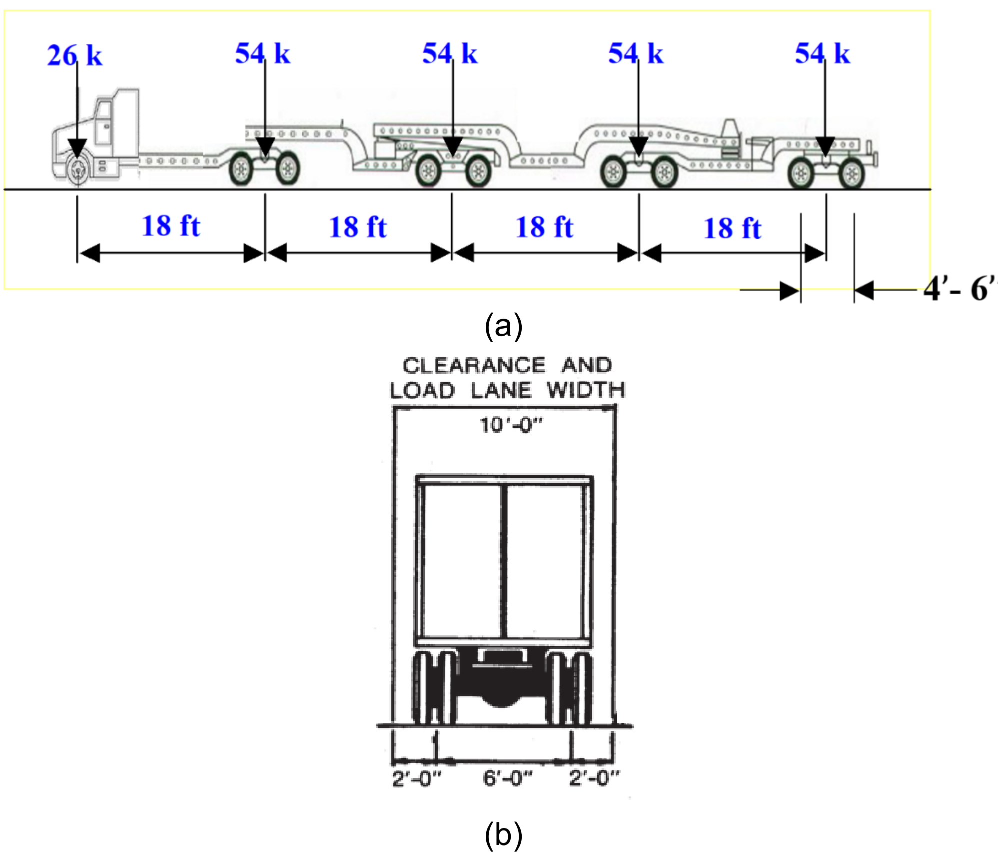

Figure 3.6.1.4.1-2 — Permit Truck, P9

3.6.1.4.2—Frequency

Add the following as the last 2 paragraphs:

All bridges shall be designed for load-induced infinite fatigue life as specified in Fatigue I Limit State. If the Caltrans approved $ADTT_{SL}$ is less than the 75-year $(ADTT)_{SL}$ as specified in Table 6.6.1.2.3-2, then a live load factor of 0.8 and nominal fatigue resistance as specified in Eq. (6.6.1.2.5-2) shall apply.

$(ADTT)_{SL}$ shall be taken as 20, for the Fatigue II limit state.

C 3.6.1.4.2

Add the following as the last paragraph:

An $(ADTT)_{SL}$ of 2500 for the design fatigue truck as specified in Article 3.6.1.4.1 has been successfully used for designing new structures and widenings in California. Since the number of stress cycles caused by an *ADTT* of 2500 is greater than that caused by a 75-year $(ADTT)_{SL}$ satisfying infinite life, all bridges are designed for load-induced infinite fatigue life as specified in Fatigue I Limit State. Based on variation of sizes, weights and volumes of P5 through P13 Permit trucks operating in California, along with a growth rate of 1% for a 75-year design life, the volumes of P5 through P13 trucks are conservatively converted to an equivalent fatigue permit truck (P9) with a traffic volume of *ADTT* = 20.

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3.6.1.6—Pedestrian Loads

Replace the article with the following:

A pedestrian load of 0.075 ksf shall be applied to all sidewalks wider than 2.0 ft and considered simultaneously with the vehicular design live load in the vehicle lane. Where vehicles can mount the sidewalk, sidewalk pedestrian load shall not be considered concurrently. If a sidewalk may be removed in the future, the vehicular live loads shall be applied at 1.0 ft from edge-of-deck for design of the overhang, and 2.0 ft from edge-of-deck for design of all other components.

Bridges intended for only pedestrian, equestrian, light maintenance vehicle, and/or bicycle traffic shall be designed in accordance with AASHTO's *LRFD Guide Specifications for the Design of Pedestrian Bridges*.

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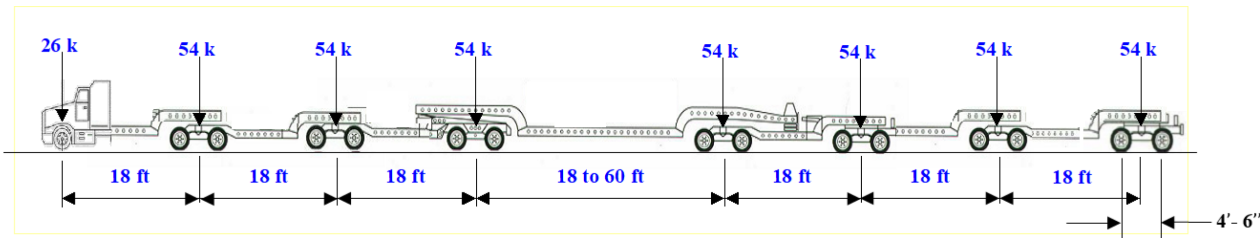
Add Article 3.6.1.8 as follows:

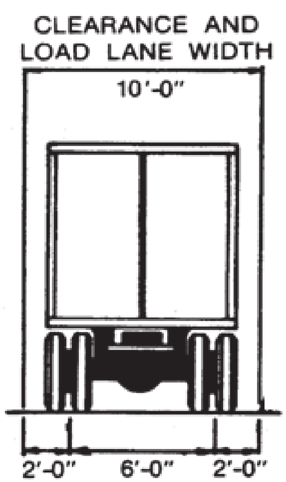
3.6.1.8—Permit Vehicle: LL_{permit}

3.6.1.8.1—General

The weights and spacings of axles and wheels for the design permit truck, P15, shall be as specified in Figure 3.6.1.8.1-1.



(a)



(b)

Figure 3.6.1.8.1—1 Permit Truck, P15

3.6.1.8.2—Application

The permit design live load shall be applied in combination with other loads as specified in Article 3.4.1. Axles that do not contribute to the extreme force effect under consideration shall be neglected.

Add the commentary as follows:

C3.6.1.8

Permit design live loads, or P-loads, are special design vehicular loads.

a) Apply to superstructure design with the load distribution factors from tables in Article 4.6.2.2.
b) Apply to superstructure design when the lever rule is called for by the tables in Article 4.6.2.2, for substructure design, and whenever a whole number of traffic lanes is to be used. Live loads shall be placed in the controlling of one or two separate lanes chosen to create the most severe conditions.

Dynamic load allowance shall be applied as specified in Article 3.6.2.

Multiple presence factors shall be applied as specified in Article 3.6.1.1.2. Multiple presence is already considered in the load distribution factor tables in Articles 4.6.2.2. However, the multiple presence factor for one loaded lane shall be 1.0 for the lever rule, substructures, and whenever a whole number of traffic lanes is applied.

Centrifugal force shall be applied as specified in Article 3.6.3.

3.6.2—Dynamic Load Allowance: *IM*

3.6.2.1—General

Replace the 1st paragraph with the following:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck, design tandem, or permit vehicle, other than centrifugal and braking forces, shall be increased by the percentage specified in Table 3.6.2.1-1 for dynamic load allowance.

Replace Table 3.6.2.1-1 with the following:

Table 3.6.2.1-1—Dynamic Load Allowance, *IM*

Component	*IM*
Deck Joints—All Limit States	75%
All Other Components • Fatigue and Fracture Limit State • Strength II Limit State • All Other Limit States	 15% 25% 33%

Add a new bullet to the 5th paragraph as follows:

- Non-integral abutments with elastomeric bearings between the superstructure and abutment seat.

C3.6.2.1

Replace the 4th paragraph with the following:

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 3.6.2.1-1 is the product of 4/3 and the basic 25 percent. California removed the 4/3 factor for Strength II because lane load isn't a part of the design permit vehicle used. Furthermore, force effects due to shorter permit vehicles approach those due to the HL-93. The HL-93 tandem*1.33 + lane load generally has a greater force effect than that due to the permit vehicle on short-span bridges.

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Replace the 6th paragraph with the following:

A study of dynamic effects presented in a report by the Calibration Task Group (*Nowak 1992*) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

Replace the 7th paragraph with the following:

This Article recognizes the damping effect of soil when in contact with some buried structural components, such as footings. To qualify for relief from impact, the entire component must be buried. Integral abutments including strutted abutments do not qualify for relief from impact. For the purpose of this Article, a retaining type component is considered to be buried to the top of the fill.

3.6.3—Centrifugal Forces: *CE*

Replace the 1st paragraph with the following:

For the purpose of computing the radial force or the overturning effect on wheel loads, the centrifugal effect on live load shall be taken as the product of the axle weights of the design truck, design tandem, or permit vehicle and the factor C, taken as:

(no change to equation)

Replace the 2nd paragraph with the following:

Highway design speed shall not be taken to be less than the value specified in the current edition of the Caltrans *Highway Design Manual*, or as otherwise directed. The design speed for permit vehicles shall be 25 mph, maximum.

Replace the 4th paragraph with the following:

For single column bents, centrifugal forces shall be applied horizontally at a distance 6.0 ft above the roadway surface. Otherwise, they shall be applied at the roadway surface. A load path to carry the radial force to the substructure shall be provided.

C3.6.3

Replace the 4th paragraph with the following:

Centrifugal force causes an overturning effect on the wheel loads when the radial force is applied 6.0 ft above the top of the deck. Thus, centrifugal force tends to cause an increase in the vertical wheel loads toward the outside of the bridge and an unloading of the wheel loads toward the inside of the bridge. The effect is more significant on structures with single column bents, but can be ignored for most applications. Superelevation helps to balance the overturning effect due to the centrifugal force and this beneficial effect may be considered. The effects due to vehicle cases with centrifugal force effects included should be compared to the effects due to vehicle cases with no centrifugal force, and the worst case selected.

3.6.4—Braking Force: *BR*

Replace the 2nd paragraph with the following:

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

C3.6.4

Replace 1st paragraph with the following:

Based on energy principles, and assuming uniform deceleration, the braking force determined as a fraction of vehicle weight is:

$$b = \frac{v^2}{2ga} \qquad \text{(C3.6.4-1)}$$

The overturning effect from braking is dependent on the number of axles and location of the drive train. This load may be applied at deck level with negligible effect on member sizes and quantities.

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3.6.5—Vehicular Collision Force: *CT*

3.6.5.1—Protection of Structures

Replace the 2nd paragraph with the following:

Where the design choice is to provide structural resistance, the pier or abutment shall be designed for an equivalent static force of 600 kips, which is assumed to act in any direction, in a horizontal plane, at a distance of 5.0 ft above ground. The flexural capacity may be based on the idealized plastic moment of the loaded component as defined in the *Caltrans Seismic Design Criteria*. Shear shall also be investigated.

C3.6.5.1

Add a new paragraph to the beginning of the commentary:

In general, abutments do not need to be investigated for this loading condition. Bin abutments should be investigated for vehicular collision force.

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3.7.5—Change in Foundations Due to Limit State for Scour

Replace the article with the following:

The provisions of Article 2.6.4.4 shall apply. The potential effects due to the percentages of channel degradation or aggradation, contraction scour, and local scour shall be considered in the limit states shown in Table 3.7.5-1.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered as specified in Section 2, and Articles 3.4.1 and 10.5 of the Specifications and California Amendments.

C3.7.5

Replace the 2nd paragraph with the following:

Provisions concerning the effects of scour are given in Section 2. Scour is not a force effect per se, but by changing the conditions of the substructure it may significantly alter the consequences of force effects acting on structures. The design for fully-factored live loads in the scour conditions described for the strength limit state is in lieu of designing for an extreme event for flood.

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3.8.1.3—Wind Load on Live Load: *WL*

Replace the 1st paragraph with the following:

Wind load on live load shall be represented by a continuous force of 0.10 klf acting transverse to the roadway and shall be transmitted to the structure. For single column bents *WL* shall be applied horizontally at a distance 6.0 ft above the roadway surface. Otherwise, it shall be applied at the roadway surface.

C3.8.1.3

Add a new 3rd paragraph as follows:

Force effects due to this overturning couple of the vehicle are negligible in structures on piers and multi-column bents, and can be ignored for most applications.

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3.10—EARTHQUAKE EFFECTS: EQ

Add a new paragraph as follows:

All provisions for seismic analysis, design, and detailing of bridges contained in Article 3.10 and elsewhere shall be superseded by the *Caltrans Seismic Design Criteria* or *Caltrans Seismic Design Specifications for Steel Bridges* or both.

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3.12.2—Uniform Temperature

Replace the article with the following:

The design thermal movement associated with a uniform temperature change shall be calculated using Procedure A.

3.12.2.1—Temperature Range for Procedure A

Replace the 1st paragraph with the following:

The ranges of temperature shall be as specified in Table 3.12.2.1-1. Half the difference between the extended lower and upper boundary shall be used to calculate force effects due to thermal deformation. Force effects calculated using gross section properties shall use the lower value for γ_{TU}.

Replace the 2nd paragraph with the following:

The minimum and maximum temperatures specified in Table 3.12.2.1-1 shall be taken as $T_{MinDesign}$ and $T_{MaxDesign}$ respectively, in Eqs. 3.12.2.1-1 and 3.12.2.3-1.

Add a 3rd paragraph as follows:

The design thermal movement range, Δ_T, for force effects in structural analysis shall be investigated for the following:

$$\Delta_T = \pm \frac{\alpha L\left(T_{MaxDesign} - T_{MinDesign}\right)}{2} \qquad (3.12.2.1\text{-}1)$$

where:

L = expansion length, the distance from the point of no thermal movement to the point under consideration (in.)

α = coefficient of thermal expansion (in./in./°F)

3.12.2.2—Temperature Range for Procedure B

Delete the entire article and commentary.

3.12.2.3—Design Thermal Movements

Replace the article with the following:

The design thermal movement range, Δ_T, for joints and bearings, shall depend upon the extreme bridge design temperatures defined in Article 3.12.2.1 or site specific air temperature data and be determined as:

$$\Delta_T = \alpha L\left(T_{MaxDesign} - T_{MinDesign}\right) \quad (3.12.2.3\text{-}1)$$

where:

L = expansion length (in.)
α = coefficient of thermal expansion (in./in./°F)

Add a new commentary as follows:

C3.12.2.3

The designer should make appropriate allowances for avoiding the possibility of hard surface contact between major structural components. Such conditions include the contact between slotted holes and anchor bolts, and between girders and abutments. Expansion joint and bearing design should account for differences between the setting temperature and an assumed design installation temperature. Refer to Section 14 for additional design requirements for joints and bearings.

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4.3—NOTATION

Add the following notations:

I_{eff} = effective moment of inertia of the section, transformed to concrete (in.^{4}) (C4.5.2.2), (C4.5.2.3)
I_{gs} = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4}) (C4.5.2.2), (C4.5.2.3)

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4.4—ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

Delete the 3rd paragraph.

C4.4

Delete the last paragraph.

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4.5.2.3—Inelastic Behavior

C4.5.2.3

Add the following paragraph to the end of the commentary:

For cast-in-place reinforced concrete columns supporting non-segmental bridge structures, engineers may use an estimated effective moment of inertia for the respective superstructure and column sections. The effective properties may be incorporated into the structural models to analyze non-seismic force demands. Engineers may use 50% of gross moment of inertia of column (ignoring rebar) as effective moment of inertia ($I_{eff} = I_{gr} / 2$).

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4.6.1.1—Plan Aspect Ratio

Replace the last bullet with the following:

- The length-to-width restriction specified above does not apply to concrete box girder bridges.

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4.6.2.2—Beam-Slab Bridges

4.6.2.2.1—Application

Replace the 1st paragraph with the following:

The provisions of this Article may be applied to superstructures modeled as a single spine beam for straight girder bridges and horizontally curved concrete bridges, as well as horizontally curved steel girder bridges complying with the provisions of Article 4.6.1.2.4. The provisions of this Article may also be used to determine starting point for some methods of analysis to determine force effects in curved girders of any degree of curvature in plan.

Replace the 6th paragraph with the following:

Bridge not meeting the requirements of this article shall be analyzed as specified in Article 4.6.3, or as directed by the Owner.

C4.6.2.2.1

Add a new 1st paragraph as follows:

The distribution factor method may be used when the superstructure model is analyzed as a spine beam in 1-D, 2-D, or 3-D space.

Add the following after the 8th paragraph of the article:

For curved bridges having large skews (> 45°), the designer shall consider a more refined analysis that also accounts for torsion.

Add the following at the end of the 9th paragraph of the article:

Cast-in-place multicell concrete box girder bridge types may be designed as whole-width structures. Such cross-sections shall be designed for the live load distribution factors in Articles 4.6.2.2.2 and 4.6.2.2.3 for interior girders, multiplied by the number of girders, i.e., webs. The live load distribution factors for moment shall be applied to maximum moments and associated moments. The live load distribution factor for shear shall be applied to maximum shears and associated shears.

Replace the article title for *4.6.2.2.2b* with the following:

4.6.2.2.2b-i—Interior Beams with Concrete Decks

Add a new article as follows:

4.6.2.2.2b-ii—Monolithic one- and two-Cell Boxes

For cast-in-place concrete box girders shown as cross-section type “d”, the live load distribution for moment on one-cell and two-cell (N_c = 1 & 2) boxes shall be specified in terms of whole-width analysis. Such cross-sections shall be designed for the total live load lanes specified in Table 4.6.2.2.2b-2 where the moment reinforcement shall be distributed equally across the total bridge width within the effective flanges.

Add a new commentary as follows:

C4.6.2.2.2b-ii

Chung, et al (2008) conducted parametric studies on one-cell and two-cell box girder bridges using 3-D analysis. The equations for the total live load lanes are applicable to box girders that meet the following conditions:

- Equal girder spacing,
- $0.04 \leq \frac{d}{12L} \leq 0.06$
- Deck overhang length < $0.5S$

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Add the following table after Table 4.6.2.2.2b-1:

Table 4.6.2.2.2b-2—Total Design Live Load Lanes for Moment

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Total Live Load Design Lanes	Range of Applicability
Cast-in-Place Concrete Multicell Box	d	One-Cell Box Girder	$60 < L < 240$ $35 < d < 110$ $N_c = 1$
		Up to One Lane Loaded* $\frac{W}{12}(1.65\text{-}0.01W)$**	$6 \le W < 10$
		1.3	$10 \le W \le 24$
		Any Fraction or Number of Lanes: $\frac{W}{12}(1.65\text{-}0.01W)$**	$6 \le W < 12$
		$\frac{W}{12}(1.5\text{-}0.014W)$	$12 \le W < 20$
		2.1	$20 \le W \le 24$
		Two-Cell Box Girder	$60 < L < 240$ $35 < d < 110$ $N_c = 2$
		Up to One Lane Loaded*: $1.3 + 0.01\,(W\text{-}12)$	$12 \le W \le 36$
		Any Fraction or Number of Lanes: $\frac{W}{12}(1.5\text{-}0.014W)$	$12 \le W \le 36$

* Corresponds to one full truck, two half trucks, or one half truck wheel load conditions.

** For $6 \le W < 10$, the equation applies to bridge widen structures that have positive moment connections to the existing bridges.

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4.6.2.2.2e—Skewed Bridges

Delete the 1st paragraph and Table 4.6.2.2.2e-1.

C4.6.2.2.2e

Replace the 1st paragraph with the following:

Caltrans does not take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.

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Replace the article title for *4.6.2.2.3a* with the following:

4.6.2.2.3a-i—Interior Beams

Add a new article as follows:

4.6.2.2.3a-ii—Monolithic one- and two-Cell Boxes

For cast-in-place concrete box girders shown as cross-section type “d”, the live load distribution for shear on one-cell and two-cell (N_c = 1 & 2) boxes shall be specified in terms of whole-width analysis. Such cross-sections shall be designed for the total live load lanes specified in Table 4.6.2.2.3a-2 where the shear reinforcement shall be equally distributed to each girder web for non-skew conditions.

Add a new commentary as follows:

C4.6.2.2.3a-ii

Chung et. al. (2008) conducted parametric studies on one-cell and two-cell box girder bridges using 3-D analysis. The equations for the total live load lanes are applicable to box girders that meet the following conditions:

- Equal girder spacing,
- $0.04 \leq \frac{d}{12L} \leq 0.06$
- Deck overhang length < 0.5S

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Add the following table after Table 4.6.2.2.3a-1:

Table 4.6.2.2.3a-2—Total Design Live Load Lanes for Shear

<table>
<tr><th>Type of Superstructure</th><th>Applicable Cross-Section from Table 4.6.2.2.1-1</th><th>Total Live Load Design Lanes</th><th>Range of Applicability</th></tr>
<tr><td rowspan="2">Cast-in-place Concrete Multicell Box</td><td rowspan="2">d</td><td>One-Cell Box Girder
$2\left(\frac{S}{4}\right)^{0.4}\left(\frac{d}{12L}\right)^{0.06}$</td><td>$60 < L < 240$
$35 < d < 110$
$N_c = 1$
$6 \le S \le 14$</td></tr>
<tr><td>Two-Cell Box Girder
$3\left(\frac{S}{4.8}\right)^{0.5}\left(\frac{d}{12L}\right)^{0.09}$</td><td>$60 < L < 240$
$35 < d < 110$
$N_c = 2$
$6 \le S \le 14$</td></tr>
</table>

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4.6.2.2.3c—Skewed Bridges

Replace the 2nd paragraph with the following:

In determining the end shear in bridges with typical cross section type g (as shown in table 4.6.2.2.1-1), the skew correction at the obtuse corner shall be applied to all the beams.

Replace Table 4.6.2.2.3c-1 with the following:

Table 4.6.2.2.3c-1—Correction Factors for Live Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-section	a, e, k and also i, j if sufficiently connected to act as a unit	For exterior girder: $1.0+0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$ For first interior girder of T-Sections: $1.0+\left(0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta\right)/6$	$0° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Cast-in-place Concrete Multicell Box	d	For exterior girder: $1.0+\frac{\theta}{50}\le 1.6$ For first interior girder: $1.0+\frac{\theta}{300}\le 1.1$	$0° < \theta \le 60°$ $6.0 < S \le 13.0$ $20 \le L \le 240$ $35 \le d \le 110$ $N_c \ge 3$
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0+\frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan\theta$	$0° < \theta \le 60°$ $6.0 \le S \le 11.5$ $20 \le L \le 140$ $18 \le d \le 65$ $N_b \ge 3$
Concrete Box Beams Used in Multibeam Decks	f, g	$1.0+\frac{12.0L}{90d}\sqrt{\tan\theta}$	$0° < \theta \le 60°$ $20 \le L \le 120$ $17 \le d \le 60$ $35 \le b \le 60$ $5 \le N_b \le 20$

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4.6.2.2.5—Special Loads with Other Traffic

Replace the 1st paragraph with the following:

Except as specified herein, the provisions of this article may be applied where the approximate methods of analysis for beam-slab bridges specified in Article 4.6.2.2 and slab-type bridges specified in Article 4.6.2.3 are used. The provisions of this article shall not be applied where:

- the lever rule has been specified for both single lane and multiple lane loadings, or
- the special requirement for exterior girders of beam-slab bridge cross-sections with diaphragms, specified in Article 4.6.2.2.2d has been utilized for simplified analysis, or
- two identical permit vehicles in separate lanes are used, as specified in CA amendment to Article 3.4.1 and 3.6.1.8.

Add the following new articles:

4.6.2.2.6—Permanent Loads Distribution

4.6.2.2.6a— Structural Element Self-Weight

Except for cast-in-place concrete box girder bridges, shears and moments due to the structural section self-weight shall be distributed to individual girders by the tributary area method.

For cast-in-place concrete multi-cell boxes (d), the shears in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed. The correction factors are applied to individual girder shears and are obtained from Table 4.6.2.2.6a-1. The correction factors should be applied between the point of support at the obtuse corner and mid-span, and may be decreased linearly to a value of 1.0 at mid-span, regardless of end condition. This factor should not be applied in addition to modeling skewed supports.

For cast-in-place concrete Tee Beams (e), the shears in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed. The shear correction factors are applied to individual girders and are obtained similarly to live load shears in Article 4.6.2.2.3c-1.

Table 4.6.2.2.6a-1—Correction Factors for Dead Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Cast-in-place Concrete Multicell Box	d	For exterior girder: $1.0 + \frac{\theta}{25} \leq 2.2$ For first interior girder: $1.0 + \frac{\theta}{150} \leq 1.2$	$0^\circ < \theta < 60^\circ$ $6.0 < S < 13.0$ $20 < L < 240$ $35 < d < 110$ $N_c > 3$

4.6.2.2.6b—Non-Structural Element Loads

Non-structural loads are appurtenances, utilities, wearing surface, futures overlays, and earth cover. Curbs and wearing surfaces, if placed after the slab has been cured, may be distributed equally to all roadway stringers or beams. Barrier loads may be equally distributed to all girders. Significant loads such as barriers with concrete or masonry soundwalls, or heavy utilities, shall not be distributed equally. For box girder bridges, the non-structural element shears in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed. The correction factors are applied to individual girder shears and they are obtained similarly to permanent load shears in Article 4.6.2.2.6a.

4.6.2.2.6c—Other Loads

For cast-in-place concrete multi-cell boxes (d), the shears due to secondary prestress, creep, shrinkage, and temperature loads in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed. The correction factors are applied to individual girder shears and are obtained from Table 4.6.2.2.6a-1. The correction factors should be applied between the point of support at the obtuse corner and mid-span and may be decreased linearly to a value of 1.0 at mid-span, regardless of end condition. This factor should not be applied in addition to modeling skewed supports.

4.6.2.3—Equivalent Strip Widths for Slab-Type Bridges

Delete the 4th paragraph.

C4.6.2.3

Add a new paragraph after the 1st paragraph:

Caltrans does not take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.

4.6.2.5—Effective Length Factor, *K*

Replace the 1st paragraph with the following:

Physical column or compression member lengths shall be multiplied by an effective length factor, *K*, to compensate for rotational and translational boundary conditions other than pinned ends.

Add the following after the 2nd paragraph:

The effective length factor, *K*, of the top chord of an unbraced through truss shall be determined by considering a column with elastic lateral supports at the panel points. The contribution of the connection stiffness between the floorbeam and the vertical member shall be considered in determining the stiffness of the elastic lateral supports.

C4.6.2.5

Replace the 2nd paragraph with the following:

K is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. *KL* represents the length between inflection points of a buckled column influenced by the restraint against rotation and translation of column ends. Theoretical values of *K*, as provided by the Structural Stability Research Council, are given in Table C4.6.2.5-1 for some idealized column end conditions. For compression chord members and for web members of trusses, a *K* value equal to 1.0 and 0.85 respectively may be used conservatively based on the assumptions that no restraint would be supplied at the joints if all chord members reach maximum stress under the same loading conditions (Ziemian 2010).

Add the following after the 3rd paragraph:

When fixed end connections between the floorbeams and verticals are considered in the design, the *K* factor for the top chord of an unbraced through truss in the unbraced plane can be obtained from Table C4.6.2.5-1A.

Table C4.6.2.5-1A—*K* for Variations of *CL/P*$_c$ and *n*

K	*n* = 4	*n* = 6	*n* = 8	*n* = 10	*n* = 12	*n* = 14	*n* = 16
	***CL/P*$_c$**						
1.000	3.686	3.616	3.660	3.714	3.754	3.785	3.809
1.020		3.284	2.944	2.806	2.787	2.771	2.774
1.042		3.000	2.665	2.542	2.456	2.454	2.479
1.053			2.595				
1.064		2.754		2.303	2.252	2.254	2.282
1.087		2.643		2.146	2.094	2.101	2.121
1.111	3.352	2.593	2.263	2.045	1.951	1.968	1.981
1.176		2.460	2.013	1.794	1.709	1.681	1.694
1.250	2.961	2.313	1.889	1.629	1.480	1.456	1.465
1.333		2.147	1.750	1.501	1.344	1.273	1.262
1.429	2.448	1.955	1.595	1.359	1.200	1.111	1.088
1.538		1.739	1.442	1.236	1.087	0.988	0.940
1.667	2.035	1.639	1.338	1.133	0.985	0.878	0.808
1.888		1.517	1.211	1.007	0.860	0.768	0.708
2.000	1.750	1.362	1.047	0.847	0.750	0.668	0.600

where:

C = lateral stiffness of the U-frame (Figure C4.6.2.5-1A) made of the truss verticals and the floorbeam (kip/in.)

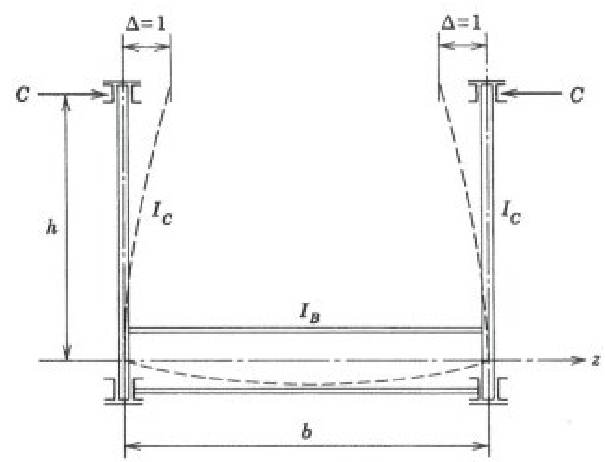

Figure C4.6.2.5-1A—U-Frame

$$C = \frac{E}{h^2[(h/3I_c)+(b/2I_B)]} \qquad \text{(C4.6.2.5-1A)}$$

L = length of the chord between panel points (in.)

P_c = maximum factored compressive load in the top chord at the Strength Limit State (kip)

n = number of truss panels in the vertical plane on one side of the bridge along its span length

Δ = lateral deflection resulting from lateral load C and shown schematically in Figure C4.6.2.5-1A (in.)

h = height of truss measured from the center line of the top chord member to the center line of the floorbeam (in.)

b = spacing between center lines of trusses (in.)

I_c = equivalent moment of inertia of truss vertical at one panel point (in.^{4})

I_B = moment of inertia of the floor beam (in.^{4})

E = modulus of elasticity of the truss material (ksi)

The K factors listed in Table C4.6.2.5-1A were developed by Holt (1952, 1956) and recommended by the Structural Stability Research Council (Ziemian 2010). Typical bridge truss proportions and transverse frame stiffness values lead to K factors less than 2.0.

In lieu of the *K* factor determined from Table C4.6.2.5-1A, the stability of the top chords of the truss may be evaluated by using a second-order numerical analysis procedure provided the following aspects are included in the model:

- A lateral out-of-plumbness of *h*/500 in the transverse direction, where *h* is the truss height
- Initial out-of-straightness of *L*/1000, both between panel points and across the entire length of the compression chord.
- Effects of the stiffness of vertical to floorbeam connections.

The out-of-plumbness of *h*/500 and the out-of-straightness of *L*/1000 are fabrication tolerances as specified in *AISC Code of Standard Practice for Steel Buildings and Bridges* (2016).

4.6.3—Refined Methods of Analysis

4.6.3.1—General

Replace the 2nd paragraph with the following:

Railings, barriers, and medians shall not be considered as structurally continuous, except as allowed for deck overhang load distribution in Article 3.6.1.3.4

C4.6.3.1

Replace the 2nd paragraph with the following:

This provision reflects the experimentally observed response of bridges. This source of stiffness has traditionally been neglected but exists and may be included, per the limits of Article 3.6.1.3.4, provided that full composite behavior is assured.

4.6.3.2—Decks

4.6.3.2.1—General

Replace the 1st paragraph with the following:

Unless otherwise specified, flexural and torsional deformation of the deck shall be considered in the analysis but vertical shear deformation may be neglected. Yield-line analysis shall not be used.

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4.9—REFERENCES

Add the following references:

Chung, P.C., Shen, Bin, Bikaee, S., Schendel, R., Logus, A., "*Live Load Distribution on One and Two-Cell Box-Girder Bridges- Draft*," Report No. CT-SAC-01, California Department of Transportation, November 2008.

Holt, E.C. 1952. "Buckling of a Pony Truss Bridge," *Stability of Bridge Chords without Lateral Bracing,* Column Research Report, No. 2. Pennsylvania State College, State College, PA.

Holt, E.C. 1956. "The Analysis and Design of Single Span Pony Truss Bridges," *Stability of Bridge Chords without Lateral Bracing,* Column Research Report, No. 3. Pennsylvania State University, State College, PA.

AISC. 2016. *Code of Standard Practice for Steel Buildings and Bridge*, ANSI/AISC 303-16, American Institute of Steel Construction, Chicago, IL.

Ziemian, R. D. (ed.). 2010. *Guide to Stability Design Criteria for Metal Structures*, 6th Ed., John Wiley and Sons, Hoboken, NJ.

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5.3—NOTATION

Replace the following notation:

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) (5.6.3.3)

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5.4.2.1—Compressive Strength

Replace the 3rd paragraph with the following:

The design concrete compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi. The design concrete compressive strength shall not be less than 3.6 ksi for all other reinforced concrete.

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5.4.4—Prestressing Steel

5.4.4.1 General

Add a new 2nd paragraph as follows:

ASTM A722 bars shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C5.4.4.1

Add a new paragraph as follows:

Galvanization of ASTM A722 bars is not permitted due to hydrogen embrittlement.

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5.4.6.2—Size of Ducts

Replace the 2nd paragraph with the following:

The size of ducts in cast-in-place concrete shall not exceed 0.5 times the least gross concrete thickness at the duct.

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5.5.3—Fatigue Limit State

5.5.3.1—General

Replace the 1st paragraph with the following:

Fatigue need not be investigated for concrete deck slabs in multigirder applications, approach slabs, slab bridges, or reinforced-concrete box culverts.

C5.5.3.1

Replace the 3rd paragraph with the following:

In determining the need to investigate fatigue, Table 3.4.1-1 specifies a load factor of 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.

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5.5.3.4—Welded or Mechanical Splices of Reinforcement

Replace the 1st paragraph with the following:

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

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5.5.4.2—Resistance Factors

Replace the 2nd bullet in the 2nd paragraph with the following:

- For tension-controlled precast prestressed concrete sections with bonded strands or tendons as specified in Article 5.6.2.1:
  normal weight concrete 1.00
  lightweight concrete 1.00

Add the following bullet under the 2nd bullet in the 2nd paragraph:

- For tension-controlled cast-in-place post-tensioned concrete sections or post-tensioned spliced precast concrete sections with bonded strands or tendons as defined in Article 5.6.2.1:
  normal weight concrete 0.95
  lightweight concrete 0.95

C5.5.4.2

Replace Figure C5.5.4.2-1 with the following:

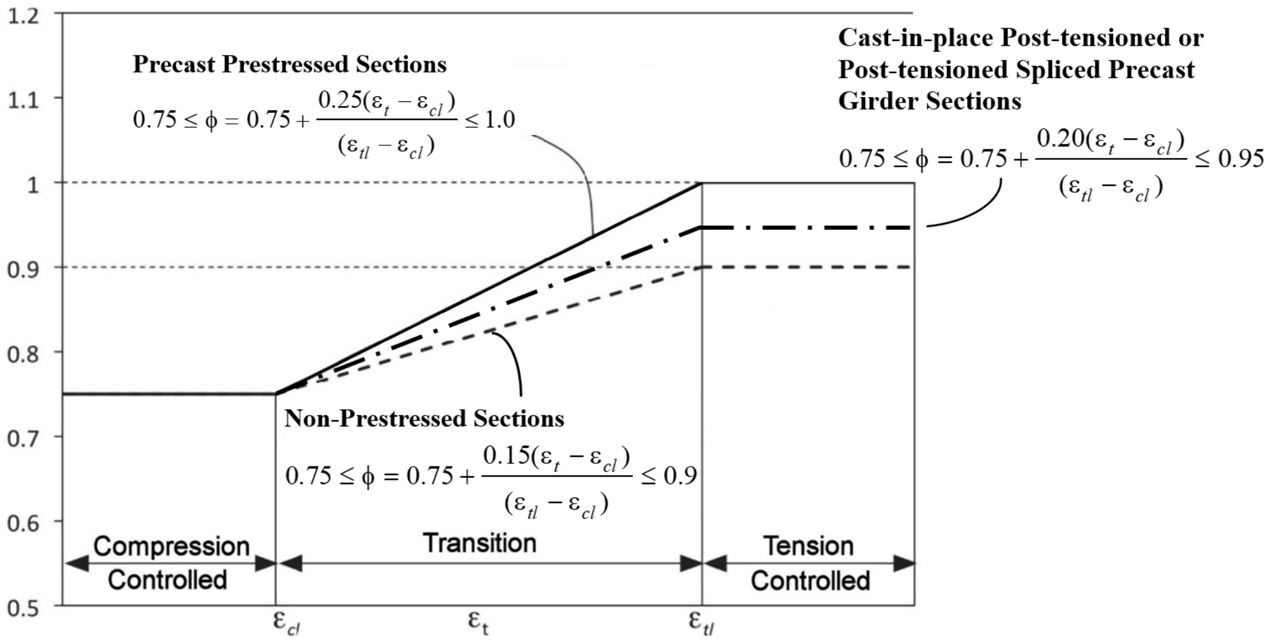

Figure C5.5.4.2-1—Variation of ϕ with Net Tensile Strain ε_t for Nonprestressed and Prestressed Sections

Replace the 4th paragraph of the article with the following:

This variation ϕ may be computed such that:

- For precast prestressed concrete sections:

$$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0 \quad (5.5.4.2\text{-}1)$$

- For cast-in-place post-tensioned or post-tensioned spliced precast concrete sections:

$$0.75 \le \phi = 0.75 + \frac{0.20(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.95 \quad (5.5.4.2\text{-}1a)$$

and for nonprestressed concrete sections such that:

$$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9 \quad (5.5.4.2\text{-}2)$$

where:

ε_t = net tensile strain in the extreme tension steel at nominal resistance (in./in.)
ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.)
ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.)

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5.5.5—Extreme Event Limit State

5.5.5.1—General

Replace the 1st paragraph with the following:

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.

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5.6.2.1—General

Replace the 11th bullet with the following:

- Sections are tension-controlled where the net tensile strain in the extreme tension steel is equal to or greater than the tension-controlled strain limit, ε_{tl}, 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 the tension-controlled strain limit constitute a transition region between compression-controlled and tension-controlled sections. For nonprestressed concrete members with factored axial compressive load less than $0.1f'_cA_g$, 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. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.005 for nonprestressed reinforcement with a specified minimum yield strength, $f_y \leq 75$ ksi and prestressed reinforcement. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.008 for nonprestressed reinforcement with a specified minimum yield strength, f_y =100 ksi. For nonprestressed reinforcement with a specified minimum yield strength between 75.0 and 100 ksi, the tension-controlled strain limit shall be determined by linear interpolation based on specified minimum yield strength.

C5.6.2.1

Replace the 5th paragraph with the following:

Where the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than the tension-controlled strain limit), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. Where 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. Members with a factored axial compressive load that is less than $0.1f'_cA_g$ can be regarded as flexural members. 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 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

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5.6.3.3—Minimum Reinforcement

Replace the following notation:

f_{cp} = 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)

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5.6.3.5.2—Deflection and Camber

Replace the 1st paragraph with the following:

Deflection and camber calculations shall consider the appropriate combinations of dead load, live load, prestressing forces, erection loads, concrete creep and shrinkage, and steel relaxation.

Add a new paragraph after the 1st paragraph:

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

C5.6.3.5.2

Replace the 1st paragraph with the following:

Camber is the deflection that is built into a member, other than by prestressing, in order to achieve the desired roadway geometry.

Add a new paragraph after the 1st paragraph:

Past experience 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.

Replace the 5th paragraph of the article with the following:

Unless a more exact determination is made, the long-term deflection of cast-in-place girder type bridges and cast-in-place slab bridges may be calculated by multiplying the instantaneous deflection values based on I_g with the following factors:

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

Alternatively, long-term deflection of cast-in-place nonprestressed concrete girder type bridges and cast-in-place nonprestressed slab bridges may be calculated by multiplying the instantaneous deflection values based on I_e with the following factor:

$$3.0 - 1.2\left(\frac{A'_s}{A_s}\right) \geq 1.6 \qquad (5.6.3.5.2\text{-}3)$$

where:

A'_s = area of compression reinforcement (in.^{2})
A_s = area of tension reinforcement (in.^{2})

Replace the last paragraph of the commentary with the following:

In prestressed concrete, the long-term deflection 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 2010), may also be used.

5.6.7—Control of Cracking by Distribution of Reinforcement

Replace the 3rd paragraph with the following:

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance, corrosion, or both. Class 2 exposure condition applies when there is increased concern of appearance, corrosion, or both.

Add a new paragraph after the 3rd paragraph:

Class 2 exposure condition applies to all bridge decks. The clear concrete cover to the top reinforcement shall be taken as 2 ½ in. to determine d_c for use in Eq. 5.6.7-1 when verifying reinforcement spacing in bridge decks.

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5.7.2—General Requirements

5.7.2.1—General

Replace the 8th paragraph with the following:

The factored shear resistance, V_r, shall be taken as:

$$V_r = \phi V_n \tag{5.7.2.1-1}$$

And the factored shear, V_u, shall be less than or equal to the factored shear resistance, V_r.

$$V_u \leq V_r \tag{5.7.2.1-1a}$$

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C5.7.2.1

Replace the last paragraph with the following:

In determining the effective web width, b_e, 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 the effective web width for the presence of ducts in fully grouted cast-in-place box girder frames.

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5.7.2.6—Maximum Spacing of Transverse Reinforcement

Replace the 1st bullet in the 1st paragraph with the following:

- If $v_u < 0.125\, f'_c$, then:

  $$S_{max} = 0.8d_v \leq 18.0\text{in.} \qquad (5.7.2.6\text{-}1)$$

C5.7.2.6

Add a new paragraph before the 1st paragraph:

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., Nov, 2007.

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5.7.3.4—Procedures for Determining Shear Resistance Parameters β and θ

C5.7.3.4

Replace the commentary with the following:

Two complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.7.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in Article 5.7.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load. In Method 2, an evaluation using tabularized values presented in Appendix B5 is adopted.

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5.7.3.4.2—General Procedure

Replace the entire article with the following:

The General Procedure for Shear Design with Tables, as described in the provisions of Appendix B5, shall be used.

C5.7.3.4.2

Delete the entire commentary.

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5.7.3.5—Longitudinal Reinforcement

C5.7.3.5

Add a new paragraph after the 1st paragraph as follows:

When using Eq. 5.7.3.5-1, 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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5.7.4.5—Computation of the Factored Interface Shear Force for Girder/Slab Bridges

Replace the last paragraph with the following:

For beams or girders, the longitudinal center-to-center spacing of non-welded interface shear connectors shall not exceed 48.0 in. or the depth of the member, *h*. For cast-in-place box girders, the longitudinal center-to-center spacing of non-welded interface shear connectors shall not exceed 18.0 in.

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5.8.4.1—Deep Components

Replace the article with the following:

Although the strut-and-tie method of Article 5.8.2 is the preferred method for designing deep components, legacy methods that have served an owner well may be used provided that all of the following are met:

- The provisions of Article 5.8.2.6 specifying the amount and spacing of crack control reinforcement are met as a minimum, except that the spacing of the crack control bars shall not exceed 24 in.;
- A limit on usable shear capacity is specified;
- The loading is placed at the appropriate depth of the component relative to the reactions in a manner consistent with the legacy method being used;
- The method of analysis reflects the distributed stress field, the behavior of cracked concrete and other nonlinear behavior anticipated at the strength or extreme event limit state.

C5.8.4.1

Add the following paragraph at the end of the commentary:

Legacy methods shall only be used for conventional geometries and loading applications.

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5.9.2.2—Stress Limitation for Prestressing Steel

Replace Table 5.9.2.2-1 with the following:

Table 5.9.2.2-1—Stress Limits for Prestressing Steel

Condition	Tendon Type		
	Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Prior to Seating --- short-term (f_{pbt})	0.90 f_{py}	0.90 f_{py}	0.90 f_{py}
Immediately prior to transfer (f_{pbt})	0.70 f_{pu}	0.75 f_{pu}	
At service limit state after all losses (f_{pe})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}
Post-Tensioning			
Maximum Jacking Stress–short term (f_{pbt})	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})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}

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5.9.2.3.2b—Tensile Stresses

Replace Table 5.9.2.3.2b-1 with the following:

Table 5.9.2.3.2b-1—Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses

Bridge Type	Location	Stress Limit
Segmental and Non-segmental Bridges	Precompressed Tensile Zone Bridges, Assuming Uncracked Sections—components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.	No tension
Other Than Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	Tension in the Precompressed Tensile Zone, 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 "non-freeze-thaw area". • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, or are located in Caltrans Environmental "freeze-thaw area". • For components with unbonded prestressing tendons	$0.19\lambda\sqrt{f'c} \leq 0.6$ (ksi) $0.0948\lambda\sqrt{f'c} \leq 0.3$ (ksi) No tension
Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	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*fy*; internal tendons or external tendons • Joints without the minimum bonded auxiliary reinforcement through joints	$0.0948\lambda\sqrt{f'c} \leq 0.3$ (ksi) No tension
	Transverse Stresses • Tension in the transverse direction in precompressed tensile zone	$0.0948\lambda\sqrt{f'c} \leq 0.3$ (ksi)
	Stresses in Other Areas • For areas without bonded reinforcement • 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 *fy*, not to exceed 30.0 ksi	No tension $0.19\lambda\sqrt{f'c}$ (ksi)

5.9.2.3.3—Principal Tensile Stresses in Webs

Replace the 1st paragraph as follows:

Except for nonsegmental cast-in-place prestressed concrete box girders with conventional geometries, the provisions specified herein shall apply to all types of post-tensioned superstructures with internal and/or external tendons. The provisions specified herein shall also apply to pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. As maximum principal tensions may not occur at the neutral axis, various locations along the height of the web shall be checked.

C5.9.2.3.3

Replace the 1st paragraph as follows:

The principal stress check is introduced to limit web cracking at the service limit state for all types of post-tensioned superstructures with internal and/or external tendons and pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. Experience has shown that the cracking in the webs of conventional pretensioned girders with a compressive strength of concrete for use in design up to 10.0 ksi and in the webs of nonsegmental cast-in-place prestressed concrete box girders with conventional geometries has not been a problem. In the context of this article, nonsegmental cast-in-place prestressed concrete box girders with unconventional geometries include, but are not limited to, any of the following:

- Variable structure depths
- Structure depths greater than 12 feet
- Span lengths greater than 300 feet
- Minimum depth-to-span ratios less than 0.045 and 0.040 for simple and continuous spans respectively for vehicular bridges
- Girder spacing to depth ratios greater than 2.0
- Individual web thicknesses less than 12 inches
- Webs with openings larger than 10% of the structure depth
- Any superstructure where a single spine beam analysis is not allowed

Replace Equation 5.9.2.3.3-4 as follows:

$$f_{\min} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) - \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$$

(5.9.2.3.3-4)

Replace Equation C5.9.2.3.3-1 as follows:

$$f_{\max} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) + \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$$

(C5.9.2.3.3-1)

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5.9.3.2.2b—Post-Tensioned Members

C5.9.3.2.2b

Add a new paragraph before the last one:

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

Replace Table 5.9.3.2.2b-1 with the following:

Table 5.9.3.2.2b-1—Friction Coefficients for Post-Tensioning Tendons

Type of Steel	Type of Duct	*K*	μ
Wire or strand	Rigid and semirigid galvanized metal sheathing Tendon Length: < 600 ft 600 ft < 900 ft 900 ft < 1200 ft > 1200 ft	 0.0002 0.0002 0.0002 0.0002	 0.15 0.20 0.25 >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

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5.9.3.2.3b—Post Tensioned Members

Replace Equation 5.9.3.2.3b-1 with the following:

$$\Delta f_{pES} = 0.5\frac{E_p}{E_{ci}} f_{cgp} \qquad (5.9.3.2.3b\text{-}1)$$

C5.9.3.2.3b

Replace Equation C5.9.3.2.3b-1 with the following:

$$\Delta f_{pES} = 0.5\frac{A_{ps}f_{pbt}(I_g+e_m^2A_g) - e_mM_gA_g}{A_{ps}(I_g+e_m^2A_g) + \frac{A_gI_gE_{ci}}{E_p}} \qquad (C5.9.3.2.3b\text{-}1)$$

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5.9.3.3—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 the lump sum value of 20 ksi.

C5.9.3.3

Add a new last paragraph:

The expressions for estimating time-dependent losses in Eq. 5.9.3.3-1 were developed for pretensioned members and should not be used for post-tensioned structures. Research performed by the University of California, 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 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 nonprestressing steel, and strength of concrete.

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5.9.4.3.3—Debonded Strands

Replace the 2nd paragraph with the following:

The number of partially debonded strands shall not exceed 33 percent of the total number of strands.

Replace the 3rd paragraph with the following:

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

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5.9.5.1.1—Post-Tensioning Ducts– Girders Straight in Plan

Replace the 2nd paragraph with the following:

Ducts less than 4-½ in. OD may be bundled together in groups, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 3.0 ft of anchorages.

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5.10.1—Concrete Cover

Replace the article with the following:

The minimum concrete cover for protection of reinforcement against corrosion due to chlorides shall be as provided in Table 5.10.1-1.

"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 1,500 ppm and/or a pH of less than or equal to 5.5 shall be considered non-corrosive in determining minimum cover from Table 5.10.1-1, but shall conform to the requirements of Article 5.14.6.

The splash zone is defined as the region from 3 ft below the Mean Lower Low Water (MLLW) elevation to 20 ft above the Mean Higher High Water (MHHW) elevation and a horizontal distance of 20 ft from the edge of water at the MHHW elevation.

Marine atmosphere includes both the atmosphere over land within 1,000 ft of ocean or marine water, and the atmosphere above the splash zone. Marine water, from corrosion considerations, is any body of water having a chloride content greater than or equal to 500 ppm.

For bundled bars, the minimum concrete cover in non-corrosive environments 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 environments, the cover shall be the same as that specified in Table 5.10.1-1, except that it shall not be less than the cover specified for bundled bars in non-corrosive environments.

C5.10.1

Replace the commentary 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 is typically less than 75 years. Therefore, concrete mix design for bridge decks has incorporated additional measures in Caltrans practice (adding polymer fibers, limiting W/C ratio, ensuring min. cementitious materials) to extend the service life of decks.

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 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.

In corrosive environments, the minimum concrete cover to prestressing steel not placed within ducts, shall be the same as that specified for reinforcement (Table 5.10.1-1), except that when epoxy-coated reinforcement is required per Table 5.10.1-1, the prestressing steel shall be epoxy-coated as specified in Standard Specifications, or the minimum concrete cover to the prestressing steel shall be increased by 1.0 inch beyond that specified in Table 5.10.1-1.

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.

The minimum concrete cover for protection of ducts in a corrosive environment shall be the same as that specified for reinforcement in Table 5.10.1-1, except that the concrete cover over the duct shall not be less than one-half the diameter of the duct; and, when epoxy-coated is required, the minimum concrete cover over the duct shall be increased by 0.50 inches beyond that specified for reinforcement in in Table 5.10.1-1, but shall not be less than one-half the diameter of the duct.

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 5.10.1-1, then a reduction in cover should only be done after a thorough investigation and research into existing state-of-practice.

In certain cases, such as the tying 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.

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.

Table 5.10.1-1—Minimum Concrete Cover to Reinforcement (in.) for 75-year Design Life

Structural Elements	Exposure condition									
	Non-Corrosive	Non-Marine			Marine					Freeze/ Thaw
	Atmosphere /soil/water	Corrosive soil above and extending down to 3 feet below the current lowest ground water elevation or 3 feet below the lowest recorded/measured ground water elevation — Chloride Concentration (ppm)			Atmosphere **(a)**	Water Permanently below MLLW level **(a), (b)**	Splash zone — Chloride Concentration (ppm)			De-icing salt, snow run-off, or snow blower spray **(a), (c), (e)**
		500-5,000 **(a)**	5,001-10,000 **(a)**	Greater than 10,000 **(a)**			500-5,000 **(a), (b)**	5,001-10,000 **(a), (b)**	Greater than 10,000 **(a), (b)**	
Footings & pile caps	3	3	4	5	3	2	2	3	3.5	2.5
Walls, columns & cast-in-place piles	2	3	4	5	3	2	2	3	3.5	2.5
Precast piles and Pile Extensions	2	2[(d)]	2[(b) (d)]	2.5[(b) (d)]	2[(d)]	2	2	2[(d)]	3[(d)]	2[(d)]
Top surface of deck slabs and top surface of slab bridges	2				2.5		2.5	2.5	2.5[(d)]	2.5
Bottom surface of deck slabs[(g)]	1.5				1.5		2	2.5	2.5[(d)]	2.5
Bottom surface of box girder bottom slabs and bottom surface of slab bridges	1.5				1.5		2	2.5	2.5[(d)]	1.5
Exposed faces of box girder webs and all other exposed girders, bent caps, diaphragms and hinged joints [(f)]	1.5				3.0		2	2.5	2.5[(d)]	3.0
Curbs & railings	1				1[(b)]		1	1	1[(d)]	1
Concrete surface not exposed to weather, soil, or water										
Bundled prestressing strands and prestressing strands with diameters larger than 0.5” Bars larger than No. 11									1.5	
Non-bundled prestressing strands with diameters of 0.5” and smaller No. 11 bars and smaller									1	

Continued on next page

Table 5.10.1-1 (Continued)—Minimum Concrete Cover to Reinforcement (in.) for 75-year Design Life

General Notes:

1. Except for the Non-corrosive Environment, all exposure conditions must meet the Supplementary Cementitious Materials (SCM) requirements of Caltrans Standard Specifications Section 90, "Concrete in Corrosive Environments" that correspond to the specific environment.
2. For protection of bundled bars, ducts, and/or pre-stressing steel, see Article 5.10.1.
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. Reinforcement shall meet at a minimum the requirements of ASTM A934 for epoxy coating.
c. Reinforcement shall meet at a minimum the requirements of ASTM A775 for epoxy coating.
d. Additional SCM specification requirements apply.
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 girders and slabs, the minimum cover shown in the table may be reduced by ½ inch (under plant 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.

5.10.8.1—General

Replace the 1st paragraph with the following:

The provisions of Articles 5.10.8.2.1a, b, c, 5.10.8.2.4, and 5.10.8.4.3a are valid for No. 11 bars or smaller, in normal weight concrete with design concrete compressive strength (f'_c) of up to 15.0 ksi and lightweight concrete up to 10.0 ksi, subject to the limitations as specified in each of these articles.

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5.10.8.2.1—Deformed Bars and Deformed Wire in Tension

Replace the 1st paragraph with the following:

The provisions of Articles 5.10.8.2.1a, b, c, may be used for No. 11 bars and smaller in normal weight concrete with a compressive strength of concrete for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi. Transverse reinforcement consisting of at least No. 3 bars at 12.0-in. centers shall be provided along the required development length where the design concrete compressive strength is greater than 10.0 ksi. The provisions of Article 5.10.8.2.1d may be used for concrete with a design compressive strength up to 10.0 ksi.

Replace the 2nd paragraph with the following:

For straight bars having a specified minimum yield strength greater than 75.0 ksi, transverse reinforcement satisfying the requirements of Article 5.7.2.5 for beams and Article 5.10.4.3 for columns shall be provided over the required development length. Confining reinforcement is not required in bridge slabs or decks.

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Add the following article:

5.10.8.2.1d—Tension Development Length Alternative Method

In lieu of Articles 5.10.8.2.1a, b, c, the modified tension development length, l_d, may be determined as specified in this Article for concrete with design compressive strength up to 10.0 ksi.

The modified tension development length, l_d, shall not be less than the product of the basic tension development length, l_{db}, and the modification factor or factors specified herein. The modified tension development length shall not be less than 12.0 in., except for development of shear reinforcement specified in Article 5.10.8.2.6.

The basic tension development length, l_{db}, in in. shall be taken as:

- For No. 11 bar and smaller...... $\dfrac{1.25A_b f_y}{\sqrt{f_c'}}$
- but not less than.................... $0.4\, d_b\, f_y$
- For No. 14 bars .... $\dfrac{2.7f_y}{\sqrt{f_c'}}$
- For No. 18 bars .... $\dfrac{3.5f_y}{\sqrt{f_c'}}$
- For deformed wire .... $\dfrac{0.95d_b f_y}{\sqrt{f_c'}}$

where:

A_b = area of bar (in.^{2})
f_y = specified yield strength of reinforcing bars (ksi)
f'_c = design compressive strength of concrete at 28 days (ksi), unless another age is specified, not exceeding 10 ksi
d_b = diameter of bar (in.)

The basic development length, l_{db}, shall be multiplied by the following factor or factors to increase l_d, as applicable:

- For top horizontal or nearly horizontal reinforcement, so placed that more than 12.0 in. of fresh concrete is cast below the reinforcement 1.4
- For lightweight aggregate concrete where f_{ct} (ksi) is specified. $\dfrac{0.22\sqrt{f_c'}}{f_{ct}} \geq 1.0$
- For all-lightweight concrete where f_{ct} is not specified.................................. 1.3
- For sand-lightweight concrete where f_{ct} is not specified 1.2

Linear interpolation may be used between all-lightweight and sand-lightweight provisions when partial sand replacement is used.

- For epoxy-coated bars with cover less than $3d_b$ or with clear spacing between bars less than $6d_b$ 1.5
- For epoxy-coated bars not covered above 1.2

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

The basic development length, ℓ_{db}, may be multiplied by the following factor or factors, to decrease l_d, where:

- Reinforcement being developed in the length under consideration is spaced laterally not less than 6.0 in. center-to-center, with not less than 3.0 in. clear cover measured in the direction of the spacing 0.8

- Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis .... $\dfrac{A_s \textit{ required}}{A_s \textit{ provided}}$
- Reinforcement is enclosed within a spiral composed of bars of not less than 0.25 in. in diameter and spaced at not more than a 4.0 in. pitch............... 0.75

Add the following commentary:

C5.10.8.2.1d—Tension Development Length Alternative Method

The provisions of Article 5.10.8.2.1d are similar to the AASHTO 6th Edition Article 5.11.2.1. The provisions are carried over to provide a simpler alternative to Articles 5.10.8.2.1a, b, c, in determining the development length for deformed bars and wires subjected to tension. In addition, the provisions give specific requirements in the determination of the development length for deformed #14 and #18 bars.

Note that ACI 318 provides two approaches for the determination of the development length in tension. The first approach is similar to what is presented in the AASHTO 6th Edition, and the second approach is similar to what is presented in the AASHTO 8th Edition.

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5.10.8.2.5a—Welded Deformed Wire Reinforcement

Replace the 4th paragraph with the following:

The basic development length of welded deformed wire reinforcement, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 5.10.8.2.1.

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5.10.8.4.3a—Lap Splices in Tension

Replace the 2nd paragraph with the following:

The tension development length, l_d, for the specified yield strength shall be taken in accordance with Article 5.10.8.2.1.

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5.12.3.3.1—General

Replace the 1st paragraph with the following:

The provisions of this Article shall apply at the service and strength limit states as applicable. Article 5.12.3.3 need not be applied to the design of multi-span bridges composed of precast girders with continuity diaphragms at bent caps.

C5.12.3.3.1

Add a new 1st paragraph as follows:

Historically, Caltrans has not explicitly designed for the requirements of 5.12.3.3 and has not experienced any negative performance issues using current design practice, standard details and construction methods for multi-span bridges composed of precast girders with continuity diaphragms at bent caps.

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5.12.5.3.8—Alternative Shear Design Procedure

Replace the entire article with the following:

Article 5.7.3 shall be used for the shear and torsion design of segmental post-tensioned box girders bridges.

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5.12.9.5.2—Reinforcement

Add the following paragraph to the end of the article:

Minimum shear reinforcement in drilled shafts shall be No. 5 hoops at 12 in. center to center spacing or equivalent spiral reinforcement, when permitted. Radial bundling of longitudinal reinforcement shall not be used in drilled shafts.

5.13—ANCHORS

5.13.1—General

Replace the last paragraph with the following:

Corrosion control shall be considered in any anchor application exposed to the elements. ASTM F1554 Grade 105 anchor bolts shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C5.13.1

Replace the last paragraph with the following:

Typical corrosion protection consists of the use of coatings or corrosion resistant materials. For adhesive anchors the manufacturer’s literature must document that the adhesive used is compatible with the type and extent of any coating used. Galvanization of ASTM F1554 Grade 105 anchor bolts is not permitted due to potential hydrogen embrittlement. These anchors should be carefully evaluated before use with applicable protective coatings conforming to ASTM F1554 Specifications.

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Replace the article including its title for 5.14.6 with the following:

5.14.6—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.

C5.14.6

Delete the entire commentary.

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APPENDIX B5—GENERAL PROCEDURE FOR SHEAR DESIGN WITH TABLES

B5.1—BACKGROUND

Replace the 1st paragraph with the following:

Appendix B5 is a complete presentation of the general procedures in LFRD Design (2007) The procedure in this Appendix utilizes tabularized values of β and θ.

B5.2—SECTIONAL DESIGN MODEL–GENERAL PROCEDURE

Replace the 3rd paragraph with the following:

Where consideration of torsion is required by the provisions of Article 5.7.2, V_u in Eqs. B5.2-3 through B5.2-5, and Eq. 5.7.2.1-1a in the California Amendment to Article 5.7.2.1, shall be replaced by V_{eff}.

For solid sections:

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \qquad \text{(B5.2-1)}$$

For hollow sections:

$$V_{eff} = V_u + \frac{T_u\, d_s}{2A_o} \qquad \text{(B5.2-2)}$$

Additionally, the cross sectional dimension of the web/girder in a hollow section shall satisfy the following:

$$\frac{V_u}{b_v d_v} + \frac{T_u}{2A_o b_e} \leq 0.474\sqrt{f_c'} \qquad \text{(B5.2-2a)}$$

CB5.2

Add after the 1st paragraph as follows:

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}.

Add the following definition to the 6th paragraph of the article:

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.

Replace the following definition in the 6th paragraph of the article:

V_u = factored shear force for the girder in Eq. B5.2-1 and for the web under consideration in Eq. B5.2-2 and Eq. B5.2-2a (kip).

Replace the 3rd paragraph of the commentary with the following:

For a hollow section/box girder, torsion introduces shear forces in the webs as well as in the top and bottom slabs. In most box girder sections, the torsional shear in interior girder webs is assumed to be negligible and is primarily resisted by exterior girders. For a hollow section/box girder, 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. B5.2-2 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 that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal. Equation B5.2-2a is added to check the cross section dimensions to prevent concrete crushing before yielding of steel stirrups.

6.4.3.1—High-Strength Structural Fasteners

6.4.3.1.1—High-Strength Bolts

Add a new 3rd paragraph as follows:

ASTM F3125 Grade A490 and Grade F2280 bolts, ASTM A354 Grade BD fasteners, and ASTM A722 bars shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C6.4.3.1.1

Replace the 2nd paragraph with the following:

Galvanizing is not an acceptable option within ASTM F3125 for Grade A490 and Grade F2280 bolts, but Grade A490 bolts may be coated with a zinc/aluminum coating in accordance with ASTM F1136 or F2833. Galvanization of ASTM F3125 Grade 490 and Grade F2280 bolts, ASTM A354 Grade BD fasteners, and ASTM A722 bars is not permitted due to potential hydrogen embrittlement.

6.4.3.3—Fasteners for Structural Anchorage

6.4.3.3.1—Anchor Rods

Replace the 1st paragraph with the following:

Anchor rods shall conform to ASTM F1554. ASTM F1554 Grade 105 anchor rods shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C6.4.3.3.1

Replace the 1st paragraph with the following:

Fasteners for structural anchorage are covered in a separate article so that other requirements for high-strength bolts are not applied to anchor rods. The term anchor rods, which is used in these Specifications, is considered synonymous with the term anchor bolts which has been used. Galvanization of ASTM F1554 Grade 105 anchor rods is not permitted due to potential hydrogen embrittlement. These rods should be carefully evaluated before use with applicable protective coatings conforming to ASTM F1554 Specifications.

6.6.1.2.5—Fatigue Resistance

Replace Table 6.6.1.2.5-2 with the following:

Table 6.6.1.2.5-2—Cycles per Truck Passage, *n*.

Longitudinal Members	
Simple Span Girders	1.0
Continuous Girders: 1) near interior support 2) elsewhere	 1.5 (HL-93) 1.2 (P9) 1.0
Cantilever Girders	5.0
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling	5.0
Trusses	1.0
Transverse Members	
Spacing > 20.0 ft	1.0
Spacing ≤ 20.0 ft	2.0

C6.6.1.2.5

Add a new last paragraph as follows:

Cycles per Permit Truck (P9) passage are evaluated by the rainflow method. The numbers of cycles induced by Permit Truck (P9) passage are somewhat similar to the cycles induced by the HL-93 truck used for Fatigue I Limit State.

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6.10.7.1.2—Nominal Flexural Resistance

Replace Eq. 6.10.7.1.2-2 with the following:

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{\frac{D_p}{D_t} - 0.1}{0.32}\right)\right]M_p$$

(6.10.7.1.2-2)

C6.10.7.1.2

Replace the 2nd paragraph with the following:

Eq. 10.7.1.2-2 defines the inelastic moment resistance as a straight line between the ductility limits D_p/D_t = 0.1 and 0.42. It gives approximately the same results as the comparable equation in previous Specifications, but is a simpler form that depends on the plastic moment resistance M_p, the yield moment resistance M_y, and the ratio D_p/D_t.

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6.10.8.2.3—Lateral Torsional Buckling Resistance

Replace C_b related Equations (6.10.8.2.3-6) and (6.10.8.2.3-7), and related symbols as follows:

- For cantilevers where the free end is unbraced:

$$C_b = 1.0 \qquad (6.10.8.2.3\text{-}6)$$

- For all other cases:

$$C_b = \frac{12.5\ M_{max}}{2.5\ M_{max} + 3\ M_A + 4\ M_B + 3\ M_C} \qquad (6.10.8.2.3\text{-}7)$$

where:

M_{max} = absolute value of maximum moment in the unbraced segment (kip-in.)
M_A = absolute value of moment at quarter point of the unbraced segment (kip-in.)
M_B = absolute value of moment at centerline of the unbraced segment (kip-in.)
M_C = absolute value of moment in three-quarter point of the unbraced segment (kip-in.)

C6.10.8.2.3

Replace the 8th (Pages 6-159) to 16th paragraphs (Page 6-162) as follows:

Equation $C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$ and $C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2$ have been used in *AISC Specification* from 1961 to 1986, and in *AASHTO LRFD Bridge Design Specifications* since 1994, respectively. Those equations are only applicable to linearly varying moment diagrams between the braced points – a condition that is rare in bridge girder design. Those equations can be easily misinterpreted and misapplied to moment diagrams that are not linear within the unbraced segment. *AISC Specification* (1993) and Caltrans BDS (2004) have adopted Eq. (6.10.8.2.3-7) originally developed by Kirby and Nethercot (1979) with slight modifications. Eq. (6.10.8.2.3-7) provides a more accurate solution for unbraced lengths in which the moment diagram deviates substantially from a straight line, such as the case of a continuous bridge girder with no lateral bracing within the span, subjected to dead and live loads.

Eq. (6.10.8.2.3-7) is applicable for doubly symmetrical sections. For singly symmetrical I-shaped sections, the following expression developed by Heldwig et al. (1997) may be used:

$$C_b = \left[\frac{12.5\ M_{max}}{2.5\ M_{max} + 3\ M_A + 4\ M_B + 3\ M_C}\right] R_m \leq 3.0 \qquad (C6.10.8.2.3\text{-}1)$$

- For single curvature bending:

$$R_m = 1.0 \qquad (C6.10.8.2.3\text{-}2)$$

- For reverse curvature bending:

$$R_m = 0.5 + 0.2\left(\frac{I_{y,top}}{I_y}\right)^2 \tag{C6.10.8.2.3-3}$$

where:

$I_{y,top}$ = moment of inertia of the flange above the geometric centroid of the section about an axis through the web (in.^{4})

I_y = moment of inertia of the entire section about an axis through the web (in.^{4})

6.10.10.4.2—Nominal Shear Force

Replace Eq. (6.10.10.4.2-8) with the following:

$$P_{2n} = F_{yrs}A_{rs} \qquad (6.10.10.4.2\text{-}8)$$

where:

A_{rs} = total area of the longitudinal reinforcement within the effective concrete deck width (in.^{2})

F_{yrs} = specified minimum yield strength of longitudinal reinforcement within the effective concrete deck width (ksi)

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6.10.11.1—Transverse Stiffeners

6.10.11.1.1—General

Replace the 2nd paragraph with the following:

Stiffeners not used as connection plates shall be welded to the compression flange and fitted tightly to the tension flange. Single-sided stiffeners on horizontally curved girders shall be attached to both flanges.

Replace the 4th paragraph with the following:

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange or longitudinal stiffener-to-web weld shall not be less than $4t_w$, nor more than $6t_w$. In no case shall the distance exceed 4.0 in.

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6.13—CONNECTIONS AND SPLICES

6.13.1—General

Replace the 1st paragraph with the following:

Except as specified herein, connections and splices for primary members subject only to axial tension or compression shall be designed at the strength limit state for not less than 100 percent of the factored axial resistance of the member or element.

Replace the 2nd paragraph with the following:

Connections and splices for primary members subjected to combined force effects, other than splices for flexural members, shall be designed at the strength limit state for not less than 100 percent of the factored axial resistance of the member determined as specified in Articles 6.8.2 or 6.9.2, as applicable.

C6.13.1

Replace the 1st paragraph with the following:

For primary members subjected to force effects acting in multiple directions due to combined loading, such as members in rigid frames, arches, and trusses, a clarification of the design requirements is provided herein related to the determination of the factored resistance of the member. Connections and splices for such members are to be designed for 100 percent of the factored axial resistance of the member. The 100 percent resistance requirement is retained to provide a minimum level of stiffness and to be consistent with past practice for the design of connections and splices for axially loaded members.

Replace the 4th paragraph of the article with the following:

Where diaphragms, cross-frames, lateral bracing, stringers, or floorbeams for straight or horizontally curved flexural members which are non-primary members are included in the structural model used to determine force effects, or alternatively, are designed for explicitly calculated force effects from the results of a separate investigation, end connections for these bracing members shall be designed for the calculated factored member force effects. Otherwise, the end connections for these members shall be designed for 75 percent of the factored resistance of the member.

6.13.6.1.3b—Flange Splices

Delete the 3rd paragraph

C6.13.6.1.3b

Delete the 3rd and 4th paragraphs

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6.13.6.1.3c—Web Splices

Replace the 1st to 3rd paragraphs with the following:

As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the following two cases:

- The smaller factored shear resistance of the web at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable.
- The portion of the smaller total factored plastic moment carried by the web.

C6.13.6.1.3c

Replace the 3rd to 4th paragraphs with the following:

Figure C6.13.6.1.3c-1 shows stress distribution of web in the plastic moment state. The portion of the flexural moment carried by the web can be expressed by the combination of a design moment, M_{uw}, and a design horizontal force resultant, H_{uw}, applied at the mid-depth of the web at the point of the splice. This horizontal force resultant may be assumed distributed equally to all web bolts.

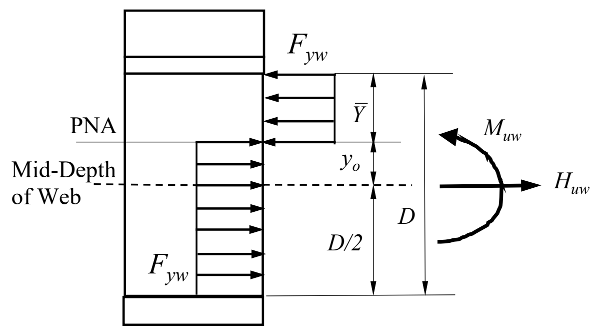

Figure C6.13.6.1.3c-1—Stress Distribution of Web at Plastic Moment State

$$M_{uw} = \phi_f \frac{t_w F_{yw}}{4}(D^2 - 4y_o^2) \quad \text{(C6.13.6.1.3c-1)}$$

$$H_{uw} = \phi_f (2t_w y_o F_{yw}) \quad \text{(C6.13.6.1.3c-2)}$$

If the plastic neutral axis is within the web,

$$y_o = \frac{D}{2} - \bar{Y} \quad \text{(C6.13.6.1.3c-3)}$$

Otherwise:

$$y_o = \frac{D}{2}$$

where:

t_w = web thickness (in.)
D = web depth (in.)
F_{yw} = specified minimum yield strength of the web at the point of splice (ksi)
y_o = distance from the mid-depth of the web to the plastic neutral axis (in.)
$\bar{Y}$ = distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located (in.)
ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

6.13.6.1.3c—Web Splices

Replace the 6th paragraph with the following:

As a minimum, bolted connections for web splices shall be checked for slip under Load Combination Service II, as specified in Table 3.4.1-1, or due to the deck casting sequence, at the point of splice, whichever governs, for the following two cases:

- Shear.
- Portion of the moment carried by the web.

C6.13.6.1.3c

Add the following after the 4th paragraph:

Figure C6.13.6.1.3c-2 shows flexural stress distribution of web under Load Combination Service II or due to deck casting sequence. The portion of the flexural moment carried by the web can be expressed by the combination of a design moment, M_{uw}, and a design horizontal force resultant, H_{uw}, applied at the mid-depth of the web at the point of the splice. This horizontal force resultant may be assumed distributed equally to all web bolts.

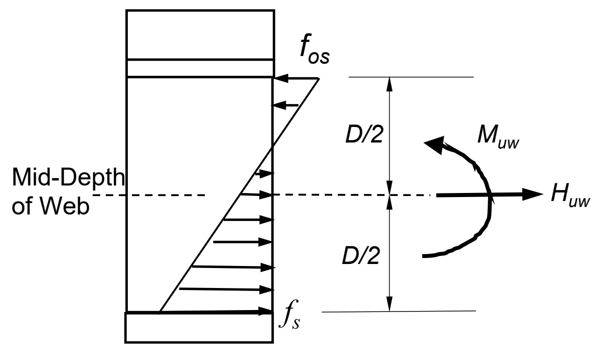

Figure C6.13.6.1.3c-2—Stress Distribution of Web at Factored Moment State

$$M_{uw} = \frac{t_w D^2}{12} |f_s - f_{os}| \qquad \text{(C6.13.6.1.3c-4)}$$

$$H_{uw} = \frac{t_w D}{2} (f_s + f_{os}) \qquad \text{(C6.13.6.1.3c-5)}$$

where:

f_s = larger flexural stress at the inner fiber of the flange under consideration for the smaller section at the point of splice (positive for tension and negative for compression) (ksi)

f_{os} = flexural stress at the inner fiber of the other flange of the smaller section at the point of splice concurrent with f_s (positive for tension and negative for compression) (ksi)

Flexural stress, f_s and f_{os} are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross-sections supporting those loadings.

6.13.6.2—Welded Splices

C6.13.6.2

Add the following as the 2nd paragraph:

Unnecessary field splices should be avoided. Welded field splices are subject to less control over the welding conditions and accessibility to the piece being welded. Additionally, access or cope holes detailed to allow for field welding activities are subjected to applied tension and/or stress reversal.

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6.14.2.8—Gusset Plates

6.14.2.8.1—General

Add the following before the 1st paragraph:

Gusset plates, fasteners, and welds connecting main members shall be designed at the strength limit state for not less than 100 percent of the factored resistances of the member.

Gusset plates, fasteners, and welds connecting other members shall be designed at the strength limit state for not less than the factored force effects of the member.

C6.14.2.8.1

Add the following after the 2nd paragraph:

Major revisions are based on Caltrans successful practice and *Caltrans Seismic Design Specifications for Seismic Design of Steel Bridges* (Caltrans 2016).

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6.14.2.8.7—Edge Slenderness

Add the following after the 1st paragraph:

For stiffened edge, the following requirements shall be satisfied:

- For welded stiffeners, slenderness ratio of the stiffener plus a width of gusset plate equal to ten times its thickness shall be $l/r \leq 40$.
- For bolted stiffeners, slenderness ratio of the stiffener between fasteners shall be $l/r \leq 40$.
- The moment of inertia of the stiffener shall be

$$I_s \geq \begin{cases} 1.83t^4\sqrt{(b/t)^2\text{-}144} \\ 9.2t^4 \end{cases} \qquad (6.14.2.8.7\text{-}2)$$

where:

I_s = moment of inertia of a stiffener about its strong axis (in.^{4})
b = width of a gusset plate perpendicular to the edge (in.)
t = thickness of a gusset plate (in.)

Add three new Articles 6.14.2.8.8 to 6.14.2.8.10 as follows:

6.14.2.8.8—Flexural Resistance

The factored flexural resistance of a gusset plate, M_r, shall be taken as:

$$M_r = \phi_f S F_y \qquad (6.14.2.8.8\text{-}1)$$

where:

S = elastic section modulus of the Whitmore's section of a gusset plate (in.^{3})
ϕ_f = resistance factor for flexural specified in Article 6.5.4.2

6.14.2.8.9—Yielding Resistance under Combined Flexural and Axial Force Effects

The Whitmore's effective area and other critical areas of a gusset plate subjected the combined flexural and axial force effects shall satisfy the following equation:

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} + \frac{P_u}{\phi A_f F_y} \leq 1.0 \qquad (6.14.2.8.9\text{-}1)$$

where:

ϕ = resistance factor for axial compression = 0.9, for axial tension yielding = 0.95
M_{ux} = factored moment about x-x axis of the gusset plate (k-in.)
M_{uy} = factored moment about y-y axis of the gusset plate (k-in.)
P_u = factored axial force (kip)
M_{rx} = factored flexural resistance about x-x axis of the gusset plate (in.^{3})
M_{ry} = factored flexural resistance about y-y axis of the gusset plate (in.^{3})
A_g = gross cross-sectional area of the Whitmore section (in.^{2})

6.14.2.8.10—Out-of-Plane Forces Consideration

For double gusset plate connections, out-of-plane moment shall be resolved into a couple of tension and compression forces acting on the near and far side plates.

C6.14.2.8.7

Replace the 1st paragraph with the following:

This Article is intended to provide good detailing practice to reduce deformations of free edges during fabrication, erection, and service versus providing an increase in the member compressive buckling resistance at the strength limit state. NCHRP Project 12-84 (Ocel, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free edge slenderness. The moment of inertia of the stiffener that is required to develop the post buckling strength of a long plate has been experimentally determined by Eq. (6.14.2.8.2-2) (AISI, 1962).

6.17—REFERENCES

Add the following references:

AISI. 1962. *Light Gage Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, DC.

Caltrans, 2016 *Caltrans Seismic Design Specifications for Seismic Design of Steel Bridges*, Second Edition, California Department of Transportation, Sacramento, CA.

Caltrans. 2004. *Bridge Design Specifications, Section 10 – Structural Steel*, February 2004. California Department of Transportation, Sacramento, CA.

AISC. 1993. Load and Resistance Factor Design Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, IL.

Kirby, P.A. and Nethercot, D.A. 1979, *Design for Structural Stability*, John Wiley & Sons Inc., New York, NY.

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A6.3.3—Lateral Torsional Buckling Resistance

Replace C_b notation with the following:

C_b = moment gradient modifier.

Delete the 4th and 5th bullets.

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9.5.2—Service Limit States

Replace the 1st paragraph with the following:

At service limit states, decks and deck systems shall be analyzed as fully elastic structures and shall be designed and detailed to satisfy the provisions of Sections 5, 6, 7, and 8. Deck slabs shall be designed for Class 2 exposure condition as specified in Article 5.6.7.

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Replace article and title with the following:

9.7.1.1—Minimum Thickness and Cover

Unless approved by the Owner, the minimum thickness of a concrete deck, excluding any provision for grinding, grooving, and sacrificial surface, shall conform to the deck design standards developed by Caltrans.

Deck reinforcement to be used in conjunction with the minimum deck thickness should also conform to the deck design standards developed by the Owner.

Minimum cover shall be in accordance with the provisions of Article 5.10.1.

C9.7.1.1

Replace the 3rd paragraph with the following:

The combinations of minimum concrete cover, concrete mix design and the need for protective coatings on reinforcement described in Article 5.10.1 are based on the results of monitoring bridges in California.

9.7.1.4—Edge Support

Replace the 2nd paragraph with the following:

Where the primary direction of the deck is transverse, no additional edge beam need be provided

9.7.2.2—Application

Replace the 1st paragraph with the following:

Empirical design of reinforced concrete decks and overhangs shall not be used.

Delete the 2nd paragraph

C9.7.2.2

Add a new 1st paragraph as follows:

The durability of empirically designed decks has not yet been proven in high ADTT applications.

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10.2—DEFINITIONS

Replace the following definition:

Casing — Steel pipe introduced during the drilling process to temporarily or permanently stabilize the soil within the drill hole. Depending on the details of construction, this casing may be fully extracted during concrete placement of a Cast-In-Drilled Hole (CIDH) concrete pile, or after grouting of a micropile, or may remain partially or completely in place, e.g., permanent casing, as part of the final pile configuration.

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10.3—NOTATION

Replace the following notations:

φ_{qp} = resistance factor for tip resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5) (10.8.3.5.1)
φ_{qs} = resistance factor for side resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5)

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10.5.2.1—General

Replace the 1st paragraph with the following:

Foundation design at the service limit state shall include:

- settlements,
- horizontal movements,
- overall stability, and
- total scour at the design flood.

10.5.2.2—Tolerable Movements and Movement Criteria

Add two paragraphs after the 3rd paragraph:

All applicable service limit state load combinations in Table 3.4.1-1 shall be used for evaluating foundation movement, including settlement, horizontal movement and rotation of foundations.

Under Service-I load combination eccentricity shall be limited to *B/*6 and *B*/4 when spread footings are founded on soil and rock, respectively.

The permissible (allowable) horizontal load for piles/shafts at abutments shall be evaluated at 0.25 inch pile/shaft top horizontal movement. Horizontal load on the pile from Service-I load combination shall be less than the permissible horizontal load.

C10.5.2.2

Add two paragraphs to the end of the article as follows:

No rotation analysis is necessary when eccentricity under Service-I load combination is limited to *B*/6 and *B*/4 or less for spread footings founded on soil and rock, respectively. Otherwise, it is necessary to establish permissible foundation movement criteria and the corresponding permissible eccentricity limits. When necessary for bridge abutments, such analysis is performed only for eccentricity in the longitudinal direction of the bridge.

The horizontal component of a battered pile's axial load may be subtracted from the total lateral load to determine the applied horizontal or lateral loads on pile foundations.

10.5.3.1—General

Replace the 2nd paragraph with the following:

The design of all foundations at the strength limit state shall consider:

- structural resistance and
- loss of lateral and axial support due to scour at the design flood event.

C10.5.3.1

Replace the 4th paragraph with the following:

The design flood for scour is defined in Article 2.6 and is specified in Article 3.7.5 as applicable at the strength limit state.

10.5.4.1—Extreme Events Design

C10.5.4.1

Replace the commentary with the following:

Extreme events include the check flood for scour, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included. *Seismic Design Criteria* (SDC) gives additional guidance regarding seismic analysis and design. Scour should be considered with extreme events as per Articles 3.4.1 and 3.7.5.

10.5.5.2.1—General

Replace the article with the following:

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5.

The foundation after scour due to the design flood shall provide adequate factored resistance using the resistance factors given in this article.

C10.5.5.2.1

Replace the commentary with the following:

Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters. When a single pile or drilled shaft supports a bridge column, reduction of the resistance factors in Articles 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5 should be considered.

Certain resistance factors in Articles 10.5.5.2.2, 10.5.5.2.3 and 10.5.5.2.4 are presented as a function of soil type, e.g., cohesionless or cohesive. Many naturally occurring soils do not fall neatly into these two classifications. In general, the terms "cohesionless soil" or “sand” may be connoted to mean drained conditions during loading, while "cohesive soil" or “clay” implies undrained conditions in the short-term. For other or intermediate soil classifications, such as clayey sand or silts, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil in the short term will be a drained, undrained, or a combination of the two strengths and select the method of computing resistance and associated resistance factor accordingly.

In general, resistance factors for bridge and other structure design have been derived to achieve a reliability index, β, of 3.5, an approximate probability of failure, P_f, of 1 in 5,000. However, past geotechnical design practice has resulted in an effective reliability index, β, of 3.0, or an approximate probability of a failure of 1 in 1,000, for foundations in general, and for highly redundant systems, such as pile groups, an approximate reliability index, β, of 2.3, an approximate probability of failure of 1 in 100 *(Zhang et al., 2001; Paikowsky et al., 2004; Allen, 2005).*

For bearing resistance, lateral resistance, and uplift calculations, the focus of the calculation is on the individual foundation element, e.g., a single pile or drilled shaft. Since these foundation elements are usually part of a foundation unit that contains multiple elements, failure of one of these foundation elements usually does not cause the entire foundation unit to reach failure, i.e., due to load sharing and overall redundancy. Therefore, the reliability of the foundation unit is usually more, and in many cases considerably more, than the reliability of the individual foundation element. Hence, a lower reliability can be successfully used for redundant foundations than is typically the case for the superstructure.

Note that not all of the resistance factors provided in this article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where adequate quantity and/or quality of data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the Caltrans *Bridge Design Specifications* (2000), dated November 2003.

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of some of the resistance factors for foundations provided in this article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

10.5.5.2.2—Spread Footings

Replace the 1st paragraph with the following:

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings.

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Replace Table 10.5.5.2.2-1 with the following:

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Nominal Resistance	Resistance Determination Method/Condition		Resistance Factor
Bearing in Compression	φ_b	Theoretical method - (*Munfakh et al., 2001*), in cohesive soils	0.50
		Theoretical method - (*Munfakh et al., 2001*), in cohesionless soil, using *CPT*	0.50
		Theoretical method - (*Munfakh et al., 2001*), in cohesionless soil, using *SPT*	0.45
		Semi-Empirical methods (*Meyerhof, 1957*), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_T	Precast concrete placed on cohesionless soil	0.90
		Cast-in-place concrete on cohesionless soil	0.80
		Cast-in-place or pre-cast concrete on cohesive soil	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

10.5.5.2.3—Driven Piles

Replace the entire article with the following:

Resistance factors for driven piles shall be selected from Table 10.5.5.2.3-1.

C10.5.5.2.3

Replace commentary with the following:

The resistance factors in Table 10.5.5.2.3-1 are calibrated to past WSD and Load Factored Design (LFD) practice.

Replace Table 10.5.5.2.3-1 with the following:

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

Nominal Resistance	Resistance Determination Method/Conditions	Resistance Factor	
Axial Compression or Tension	All resistance determination methods, all soils and rock	φ_{stat}, φ_{dyn}, φ_{qp}, φ_{qs}, φ_{bl}, φ_{up}, φ_{ug}, φ_{load}	0.70
Lateral or Horizontal Resistance	All soils and rock		1.0
Pile Drivability Analysis	Steel Piles	φ_{da}	See the provisions of Article 6.5.4.2
	Concrete Piles		See the provisions of Article 5.5.4.2
	Timber Piles		See the provisions of Articles 8.5.2.2
	In all three Articles identified above, use φ identified as “resistance during pile driving”		
Structural Limit States	Steel Piles	See the provisions of Article 6.5.4.2	
	Concrete Piles	See the provisions of Article 5.5.4.2	
	Timber Piles	See the provisions of Articles 8.5.2.2 and 8.5.2.3	

10.5.5.2.4—Drilled Shafts

Replace the entire article with the following:

Resistance factors for drilled shafts shall be selected from Table 10.5.5.2.4-1.

C10.5.5.2.4

Replace the entire commentary and with the following:

The resistance factors in Table 10.5.5.2.4-1 are calibrated to WSD and LFD practices.

The maximum value of the resistance factors in Table 10.5.5.2.4-1 are based on full-time inspection and field quality control during shaft construction. If a full time inspection and field quality control can not be assured, lower resistance factors should be used.

The mobilization of drilled shaft tip resistance is uncertain as it depends on many factors including soil types, groundwater conditions, drilling and hole support methods, the degree of quality control on the drilling slurry and the base cleanout, etc. Allowance of the full effectiveness of the tip resistance should be permitted only when cleaning of the bottom of the drilled shaft hole is specified and can be acceptably completed before concrete placement.

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Replace Table 10.5.5.2.4-1 with the following:

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Nominal Resistance	Resistance Determination Method/Conditions	Resistance Factor	
Axial Compression and Tension or Uplift	All soils, rock and IGM All calculation methods	φ_{stat}, φ_{up}, φ_{bl}, φ_{ug}, φ_{load}, φ_{upload}, φ_{qs}	0.70
Axial Compression	All soils, rock and IGM All calculation methods	φ_{qp}	0.50
Lateral Geotechnical Resistance	All soils, rock and IGM All calculation methods		1.0

10.5.5.3.2—Scour

Delete the entire article.

C10.5.5.3.2

Replace the 1st paragraph with the following:

See Commentary to Article 3.4.1, Extreme Events, and Article 3.7.5.

Replace Article 10.5.5.3.3 title with the following:

10.5.5.3.3—Other Extreme Event Limit States

Replace the 1st paragraph with the following:

Resistance factors for extreme event limit states, including the design of foundations to resist earthquake, blast, ice, vehicle or vessel impact loads, shall be taken as 1.0.

C10.5.5.3.3

Delete the entire commentary

10.6.1.1—General

Replace the 1st paragraph with the following:

Provisions of this article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and others substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength, swell or expansion potential and compressibility to support the footing loads.

C10.6.1.1

Replace the 2nd paragraph with the following:

Spread footing should not be used on soil or rock conditions that are determined to be expansive, collapsible, or too soft or weak to support the design loads, without excessive movements, or loss of stability.

10.6.1.3—Effective Footing Dimensions

Replace the 1st paragraph with the following:

For eccentrically loaded footings, a reduced effective area, $B' \times L'$, within the confines of the physical footing shall be used in geotechnical design for settlement and bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

Replace the 2nd paragraph with the following:

The reduced dimensions for an eccentrically rectangular footing shall be taken as:

$$B' = B - 2e_B \quad (10.6.1.3\text{-}1)$$

$$L' = L - 2e_L$$

where,

e_B = M_L / V = Eccentricity parallel to dimension B (ft)

e_L = M_B / V = Eccentricity parallel to dimension L (ft)

M_B = Factored moment about the central axis along dimension B (kip-ft)

M_L = Factored moment about the central axial along dimension L (kip-ft)

V = Factored vertical load (kips)

C10.6.1.3

Add the following at the end of the commentary:

For additional guidance, see Munfakh (2001) and Article 10.6.3.2.

10.6.1.4—Bearing Stress Distribution

Replace 1st paragraph with the following:

When proportioning footings dimensions to meet settlement and bearing resistance requirements at all applicable limit states, the distribution of bearing stress shall be assumed as:

- Uniform over the effective area for footing on soils, or
- Linearly varying, i.e., triangular or trapezoidal as applicable, for footings on rock

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10.6.1.6—Groundwater

Replace the last paragraph with the following:

The influences of groundwater on the bearing resistance of soil or rock, the expansion and collapse potential of soil or rock, and on the settlements of the structure should be considered. In cases where seepage forces are present, they should also be included in the analyses.

10.6.2.4.1—General

C10.6.2.4.1

Add the following after the last paragraph:

For eccentrically loaded footings on soils, replace *L* and *B* in these specifications with the effective dimensions *L′* and *B′*, respectively.

10.6.2.4.2—Settlement of Footing on Cohesionless Soils

Replace the 3rd paragraph with the following:

The elastic half-space method assumes the footing is supported on a homogeneous soil of infinite depth. The elastic settlement of spread footings, in feet, by the elastic half-space method shall be estimated as:

$$S_e = \frac{\left[q_O\left(1\text{-}v^2\right)\sqrt{A'}\right]}{144\, E_S\, \beta_Z} \qquad (10.6.2.4.2\text{-}1)$$

where:

q_O = applied vertical stress (ksf)
A' = effective area of footing (ft^2)
E_s = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of in-situ or laboratory tests (ksi)
β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)
v = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of v are not available from the results of in-situ or laboratory tests (dim)

C10.6.2.4.2

Replace the 6th paragraph with the following:

In Table 10.6.2.4.2-1, the β_z values for the flexible foundations correspond to the average settlement. The elastic settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footing, respectively. For low values of *L/B* ratio, the average settlement for flexible footing is about 85 percent of the maximum settlement near the center. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

Replace the 8th paragraph with the following:

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil moduli are difficult to obtain because the analyses are based on only single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent foundations.

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10.6.2.4.2—Settlement of Footing on Cohesionless Soils

Replace the last paragraph with the following:

In Figure 10.6.2.4.2-1, $N1$ shall be taken as $(N_1)_{60}$, Standard Penetration Resistance, N (blows/ft), corrected for hammer energy efficiency and overburden pressure as specified in Article 10.4.6.2.4.

10.6.2.4.3—Settlement of Footing on Cohesive Soils

Add the following after the 1st paragraph:

Immediate or elastic settlement of footings founded on cohesive soils can be estimated using Eq. 10.6.2.4.2-1 with the appropriate value of the soil modulus.

Add the following under Figure 10.6.2.4.3-3:

For eccentrically loaded footings, replace B/H_c with B'/H_c in Figure 10.6.2.4.3-3.

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Replace Article 10.6.2.6 title with the following:

10.6.2.6—Permissible Net Contact Stress

Replace the entire Articles 10.6.2.6.1 & 10.6.2.6.2 with the following:

The permissible net contact stress for spread footings shall be taken as the net footing bearing stress over the effective footing area due to Service-I Load Combination that results in an estimated foundation soil or rock settlement equal to the support-specific permissible settlement. Spread footings shall be located and sized such that the applied net bearing stress due to Service-I Load Combination does not exceed the support-specific permissible net contact stress.

C10.6.2.6

Replace C10.6.2.6.1 and Table C10.6.2.6.1-1 with the following:

The permissible settlement (total) for a bridge support is the maximum tolerable foundation settlement due to Service-I Load Combination in accordance with Article 10.5.2.2. The adequacy of a given footing size to satisfy the permissible settlement can be verified by performing a settlement analysis for the effective size and applied bearing stress, both based on the Service-I Load Combinations. However, in most cases, the design footing size can more conveniently be optimized by evaluating first the minimum effective size of a spread footing required to satisfy the permissible settlement criterion. This can be achieved by using support-specific permissible net contact pressures presented in a plot or table as a function of the effective footing width (*B'*) for a range of *L'*/*B'* ratios.

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10.6.3.1.2a—Basic Formulation

C10.6.3.1.2a

Replace the 4th paragraph with the following:

It should further be noted that the resistance factors provided in Article 10.5.5.2.2 were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with modest embedment of 4 feet or deeper because the load inclination factors were derived for footings without embedment.

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Replace the 5th paragraph with the following:

In practice, therefore, footings that are normal to a column with modest embedment, should omit the use of the load inclination factors.

Add a new paragraph after the 5th paragraph:

Unusual column geometry or loading configurations may require consideration be given to evaluating load inclination factors. A column that is not aligned normal to the footing bearing surface would be one example where inclination factors would be given consideration. In cases where inclinations are to be evaluated, the simultaneous application of both shape and inclination factors will result in overly conservative design, therefore using the lower of the two factors is recommended (Munfakh et al., 2001).

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10.6.3.1.2c—Considerations for Footings on Slopes

Replace the entire article with the following:

For footings bearing on or near slopes:

$$N_q = 0.0 \quad (10.6.3.1.2c\text{-}1)$$

In Equations 10.6.3.1.2a-2 and 10.6.3.1.2a-4, N_c and N_γ shall be replaced with N_{cq} and $N_{\gamma q}$, respectively, from Figures 10.6.3.1.2c-1 and 10.6.3.1.2c-2 for footings bearing on or near slopes. In Figure 10.6.3.1.2c-1, the slope stability factor, N_s, shall be taken as:

- For $B < H_s$:

$$N_s = 0 \quad (10.6.3.1.2c\text{-}2)$$

- For $B \geq H_s$:

$$N_s = \frac{\gamma H_s}{c} \quad (10.6.3.1.2c\text{-}3)$$

where:

B = footing width (ft)

H_s = height of sloping ground mass (ft)

C10.6.3.1.2c

Replace the 1st paragraph with the following:

A rational numerical approach for determining a modified bearing capacity factor, N_{cq}, for footings on or near a slope is given in Bowles (1988).

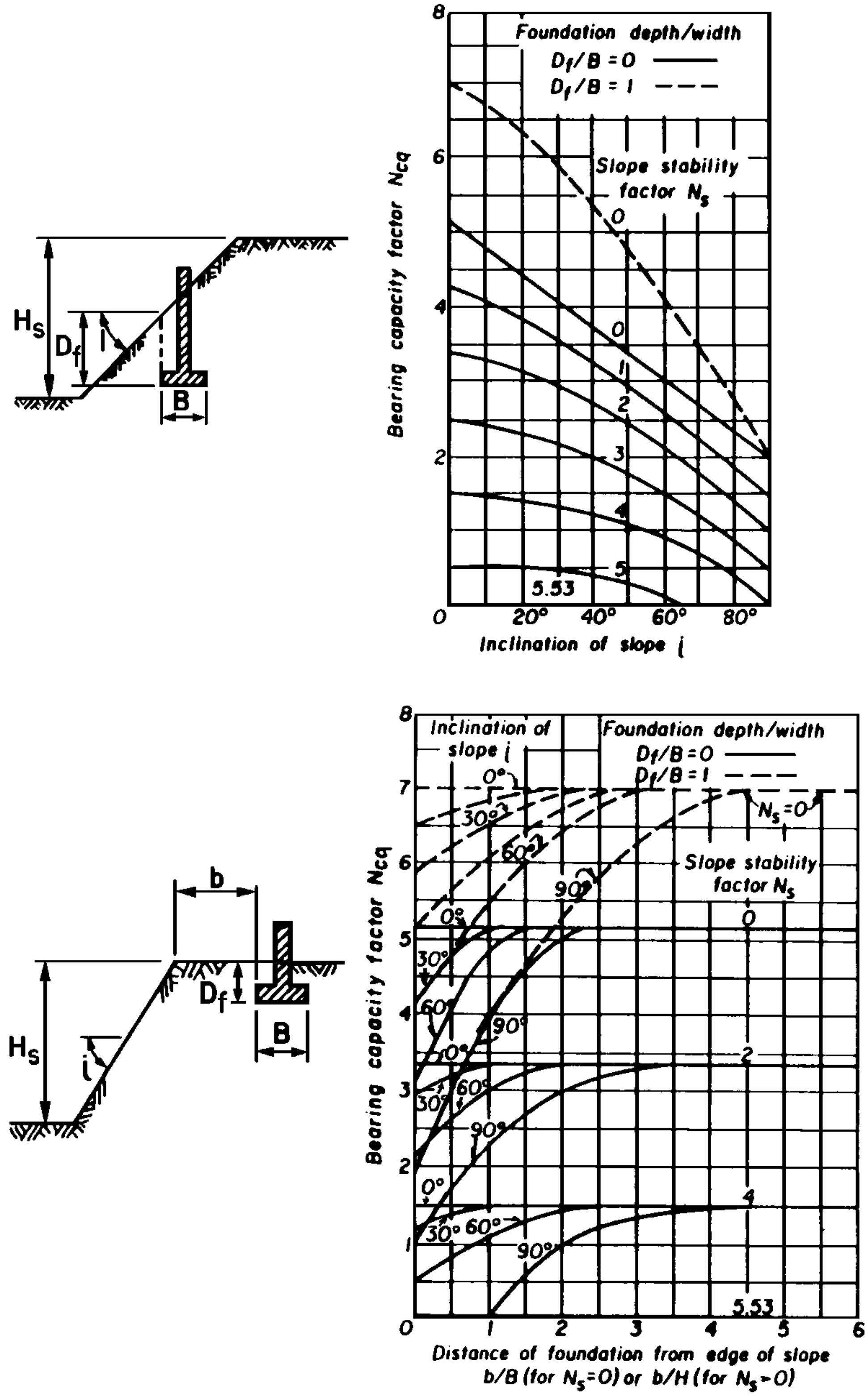
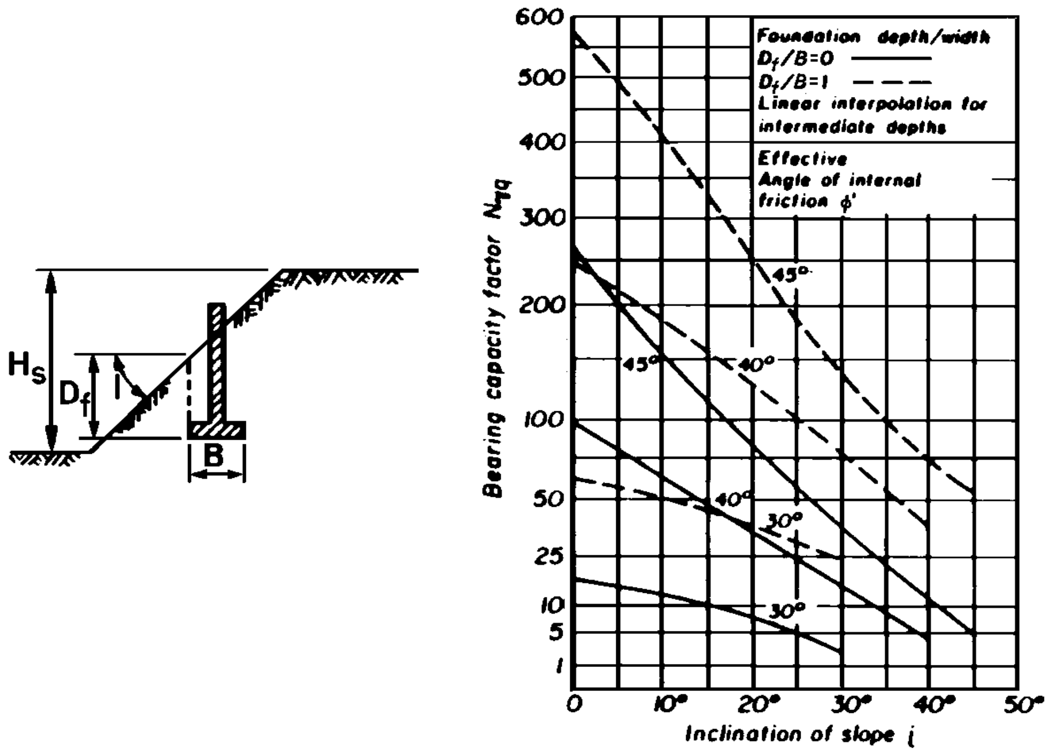


Figure 10.6.3.1.2c-1—Modified Bearing Capacity Factors for Footing in Cohesive Soils and on or adjacent to Sloping Ground after Meyerhof (1957)



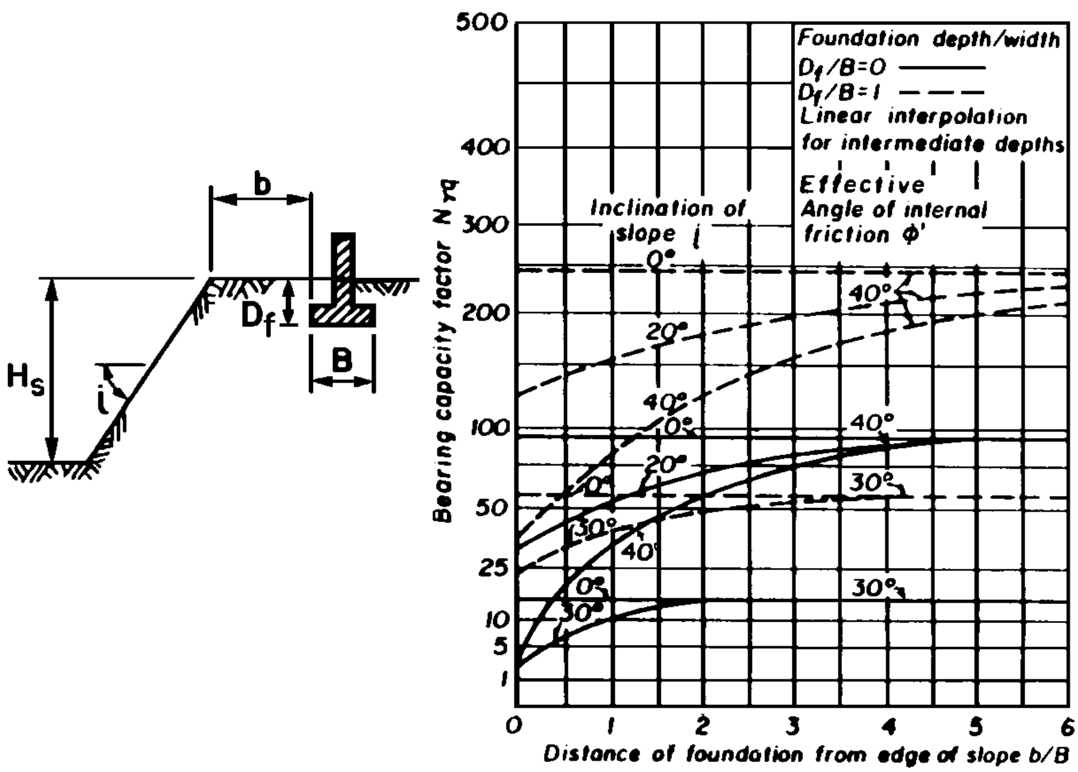


Figure 10.6.3.1.2c-2—Modified Bearing Capacity Factors for Footing in Cohesionless Soils and on or adjacent to Sloping Ground after Meyerhof (1957)

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10.6.3.1.2e—Two-Layered Soil Systems in Undrained Loading

C10.6.3.1.2e

Replace equations C10.6.3.1.2e-5 and C1.0.6.3.1.2e-6 with the following:

- For circular or square footings:

$$\beta_m = \frac{B}{4\ H_{s2}} \qquad \text{(C10.6.3.1.2e-5)}$$

$$N^*_c = 6.17$$

- For strip footings:

$$\beta_m = \frac{B}{2\ H_{s2}} \qquad \text{(C10.6.3.1.2e-6)}$$

$$N^*_C = 5.14$$

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Replace *H* with H_{s2} in Figure 10.6.3.1.2e-2,

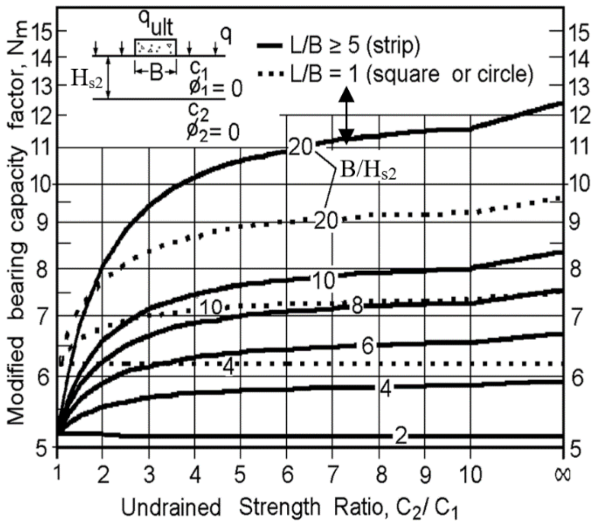

Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f—Two –Layered Soil System in Drained Loading

Replace Eq. 10.6.3.1.2f-1 with the following:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right) c'_1 \cot \varphi'_1\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\phi'_1 \left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right) c'_1 \cot \varphi'_1 \quad (10.6.3.1.2f\text{-}1)$$

C10.6.3.1.2f

Replace Eq. C10.6.3.1.2f-1 with the following:

$$q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H_{s2}}{B}} \quad (C10.6.3.1.2f\text{-}1)$$

Replace the title for 10.6.3.1.3 with the following:

10.6.3.1.3—Semiempirical Procedures for Cohesionless Soils.

C10.6.3.1.3

Add the following to the end of the commentary:

It is recommended that the *SPT* based method not be used.

10.6.3.2.1—General

C10.6.3.2.1

Replace the 1st paragraph with the following:

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the service limit state before checking nominal bearing resistance at both the strength and extreme event limit states.

Replace the article and title for *10.6.3.2.4* with the following:

10.6.3.2.4—Plate Load Test

Where appropriate, plate load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

Replace the article with the following:

The factored nominal bearing resistance of the effective footing area shall be equal to or greater than the factored bearing stress.

C10.6.3.3

Replace the commentary with the following:

Excessive differential contact stress due to eccentric loading can cause a footing to rotate excessively leading to failure. To prevent rotation, the footing must be sized to provide adequate factored bearing resistance under the vertical eccentric load that causes the highest equivalent uniform bearing stress. As any increase in eccentricity will reduce the effective area of the footing (on soil), or will increase the maximum bearing stress (on rock), bearing resistance check for all potential factored load combinations will ensure that eccentricity will not be excessive.

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10.7.1.2—Minimum Pile Spacing, Clearance, and Embedment into Cap

Replace the 1st paragraph with the following:

Center-to-center spacing shall not be less 36.0 in. or 2.0 pile diameters (whichever is greater). The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in. or 0.5 pile diameters (whichever is greater). For abutments without a pile cap, the distance from the side of any pile to the nearest edge of the abutment shall not be less than 6.0 inches.

Replace the 2nd paragraph with the following:

If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 3.0 in. into the cap for concrete piles and 5 in. into the cap for steel piles.

10.7.1.4—Battered Piles

Add the following at the end of the article:

Battered piles shall not be used at foundations of bents and piers in class S2 soil.

Battered piles may be considered for use at foundations of bents and piers in class S1 soil with approval from Owner.

10.7.1.5—Pile Design Requirements

Replace the article with the following:

Pile design shall address the following issues as appropriate:

- Pile cut off elevation, type of pile, and size and layout of pile group required to provide adequate support, with consideration of subsurface conditions, loading, constructability, and how nominal bearing pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation and estimated pile penetration to meet nominal axial resistance and other design requirements.
- Uplift, lateral loads, scour, downdrag, liquefaction, lateral spreading, and other seismic conditions.
- Foundation deflection to meet the established movement and associated structure performance criteria.
- Minimum pile penetration necessary to satisfy the requirements caused by settlement, uplift and lateral loads.
- Pile foundation nominal structural resistance.
- Pile foundation buckling and lateral stability
- Pile drivability to confirm that acceptable driving stresses and blow counts can be achieved at the nominal bearing resistance, and at the estimated resistance to reach the minimum tip elevation, if a minimum tip elevation is required, with an available driving system.
- Long-term durability of the pile in service, i.e., corrosion and deterioration.

10.7.2.2—Tolerable Movements

Replace the article with the following:

The provisions of Article 10.5.2.2 shall apply

C10.7.2.2

Replace the commentary with the following:

See Article C10.5.2.2

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10.7.2.3—Settlement

C10.7.2.3

Add the following commentary:

Since most piles are placed as groups, estimation of settlement is more commonly performed for pile groups than a single pile. The equivalent footing or the equivalent pier methods may be used to estimate pile group settlement.

The short-term load-settlement relationship for a single pile can be estimated by using procedures provided by Poulos and Davis (1974), Randolph and Wroth, (1978) and empirical load-transfer relationship or skin friction t-z curves and base resistance q-z curves. Load transfer relationships presented in API (2003) and in Article 10.8.2.2.2 can be used. Long-term or consolidation settlement for a single pile may be estimated according to the equivalent footing or pier method.

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Replace the title for 10.7.2.3.2 with the following:

10.7.2.3.2—Pile Group Settlement

Replace the 1st paragraph with the following:

Shallow foundation settlement estimation procedures in Article 10.6.2.4 shall be used to estimate the settlement of a pile group, using the equivalent footing location specified in Figure 10.7.2.3.1-1 or Figure 10.7.2.3.1-2.

Replace the 1st sentence of the 2nd paragraph with the following:

The settlement of pile groups in homogeneous cohesionless soil~~s~~ deposits not underlain by more compressible soil at deeper depth may be taken as:

Using SPT: $$\rho = \frac{qI\sqrt{B}}{N1_{60}} \qquad (10.7.2.3.2\text{-}1)$$

Using CPT: $$\rho = \frac{qBI}{2q_c} \qquad (10.7.2.3.2\text{-}2)$$

in which:

$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \qquad (10.7.2.3.2\text{-}3)$$

where:

ρ = settlement of pile group (in.)

q = net foundation pressure; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles. For friction piles, this pressure is applied at two-thirds of the pile embedment depth, D_b, in the cohesionless bearing layer. For a group of end bearing piles, this pressure is applied at the elevation of the pile tip. (ksf)
B = width or smallest dimension of pile group (ft)
I = influence factor of the effective group embedment (dim)
D' = effective depth taken as $2D_b/3$ (ft)
D_b = depth of embedment of piles in the cohesionless layer that provides support (ft)
$N1_{60}$ = SPT blow count corrected for both overburden and hammer efficiency effects (blows/ft) as specified in Article 10.4.6.2.4.
q_e = static cone tip resistance (ksf)

Replace the 4th paragraph with the following:

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width *B* below the equivalent footing.

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10.7.2.4—Horizontal Pile Foundation Movement

Replace Table 10.7.2.4-1 with the following:

Table 10.7.2.4-1—Pile P-Multipliers, P_m for Multiple Row Shading

Pile *CTC* spacing (in the Direction of Loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3
2.0*B*	0.60	0.35	0.25
3.0*B*	0.75	0.55	0.40
5.0*B*	1.0	0.85	0.70
7.0*B*	1.0	1.0	0.90

Replace the 7th paragraph with the following:

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. A P-multiplier of 1.0 shall be used for pile *CTC* spacing of 8*B* or greater. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the Figure), a P-multiplier of less than 1.0 shall only be used if the pile spacing is 4*B* or less. A P-multiplier of 0.80, 0.90 and 1.0 shall be used for pile spacing of 2.5*B*, 3*B* and 4*B*, respectively.

C10.7.2.4

Replace the 8th paragraph with the following:

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been compiled in Reese and Van Impe (2000), Caltrans (2003), Hannigan et al. (2006), and Rollins et al (2006).

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10.7.2.5—Settlement Due to Downdrag

Replace the article with the following:

The effects of downdrag, if present, shall be considered when estimating pile settlement under service limit state.

C10.7.2.5

Replace the commentary with the following:

Guidance to estimate the pile settlement considering the effects of downdrag is provided in Meyerhof (1976), Briaud and Tucker (1997) and Hennigan et al (2005).

10.7.3.1—General

Replace the article with the following:

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal bearing resistance;
- Size and configuration of the pile group to provide adequate foundation support;
- The specified pile tip elevation to be used in the construction contract document to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in Article 10.7.6;
- The maximum driving resistance expected in order to reach the specified tip elevation, including any soil/pile side resistance that will not contribute to the long-term nominal bearing resistance of the pile, e.g., surficial soft or loose soil layers, soil contributing to downdrag, or soil that will be removed by scour;
- The drivability of the selected pile to the specified tip elevation with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and
- The nominal structural resistance of the pile and/or pile group.

C10.7.3.1

Replace the 1st paragraph with the following:

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to satisfy scour concerns, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See Article 10.7.6 for additional details.

Replace the title of article 10.7.3.3 with the following:

10.7.3.3—Pile Length Estimates

Replace the article with the following:

Subsurface geotechnical information combined with static analysis methods (Article 10.7.3.8.6), preconstruction test pile programs (Article 10.7.9), and/or pile load tests (Article 10.7.3.8.2) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing resistance to establish contract pile quantities. If static analysis methods are used, potential bias in the method selected should be considered when estimating the penetration depth required to achieve the desired nominal bearing resistance. Local pile driving experience shall also be considered when making pile quantity estimates. If the depth of penetration required to obtain the desired nominal bearing, i.e., compressive, resistance is less than the depth required to meet the provisions of Article 10.7.6, the minimum penetration required per Article 10.7.6 should be used as the basis for the specified tip elevation and estimating contract pile quantities.

C10.7.3.3

Replace the 1st and 2nd paragraphs with the following:

The estimated pile length necessary to provide the required nominal resistance is determined using a static analysis, local pile driving experience, knowledge of the site subsurface conditions, and/or results from a static pile load test program. The specified pile tip elevation is often defined by the presence of an obvious bearing layer. Local pile driving experience with such a bearing layer should be strongly considered when developing pile quantity estimates.

In variable soils, a program of test piles across the site may be used to determine variable pile order lengths. Test piles are particularly useful when driving concrete piles. The specified pile tip elevation used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

Delete the 4th paragraph.

Replace the 5th paragraph with the following:

Where piles are driven to a well defined firm bearing stratum, the location of the top of the bearing stratum will dictate the pile length needed.

Delete the 6th paragraph.

Delete the 7th paragraph.

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10.7.3.4.3—Setup

C10.7.3.4.3

Replace the 3rd paragraph with the following:

If a wave equation or dynamic formula is used to determine the nominal pile bearing resistance on re-strike, care should be used as these approaches require accurate blow count measurement which is inherently difficult at the beginning of redrive (BOR).

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10.7.3.6—Scour

C10.7.3.6

Replace the 1st paragraph with the following:

The piles will need to be driven to the specified tip elevation and the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias.

Replace the 2nd paragraph with the following:

The magnitude of skin friction that will be lost due to scour may be estimated by static analysis. The static analysis used to determine the nominal axial resistance after the scour event must consider the reduction of the effective overburden stresses due to scour. Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

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10.7.3.8.1—General

Replace the article with the following:

Nominal pile bearing resistance should be field verified during pile installation using static load tests, dynamic tests, wave equation analysis, or dynamic formula. The resistance factor selected for design shall be as specified in Article 10.5.5.2.3. The production piles shall be driven to the specified tip elevation and the minimum blow count determined from the static load test, dynamic test, wave equation, or dynamic formula.

C10.7.3.8.1

Replace the commentary with the following:

This Article addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. Both the loads and resistance values are factored as specified in Articles 3.4.1 and 10.5.5.2.3, respectively, for this determination.

In most cases, the nominal resistance of production piles should be controlled by driving to the specified tip elevation and a required blow count. In a few cases, usually piles driven into cohesive soils with little or no toe resistance and very long wait times to achieve the full pile resistance increase due to soil setup, piles maybe driven to depth. However, even in those cases, a pile may be selected for testing after a sufficient waiting period, using either a static load test or a dynamic test.

10.7.3.8.2—Static Load Test

Replace the 1st paragraph with the following:

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed prior to completion of the pile set up period as determined by the Engineer. The load test shall follow the procedures specified in ASTM D 1143, and the loading procedure should follow the Quick Load Test Procedure.

C10.7.3.8.2

Replace Figure C10.7.3.8.2-1 with the following:

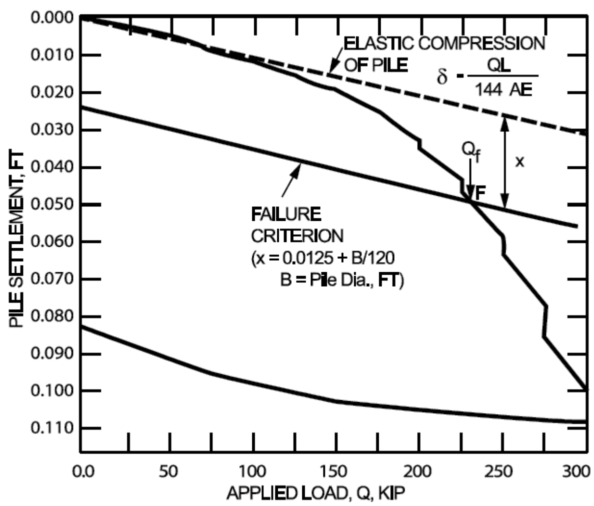

Figure C10.7.3.8.2-1—Davissons' Method for Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972).

10.7.3.8.3—Dynamic Testing

Replace the article with the following:

Dynamic testing shall be performed according to the procedures given in ASTM D 4945. If possible, the dynamic test should be performed as a re-strike test if the Engineer anticipates significant time dependent soil strength change. The pile nominal resistance shall be determined by a signal matching analysis of the dynamic pile test data if the dynamic test is used to establish the driving criteria.

Dynamic testing shall be calibrated to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.3

Replace the 1st paragraph with the following:

The dynamic test may be used to establish the driving criteria at the beginning of production driving. When dynamic testing is performed on piles up to 36 inches in diameter, a signal matching analysis (Rausche et al., 1972) of the dynamic test data should always be used to determine bearing resistance if a static load test is not performed. See Hannigan et al. (2006) for a description of and procedures to conduct a signal matching analysis. Re-strike testing should be performed if setup or relaxation is anticipated. The minimum number of piles that should be tested are as specified by the Engineer.

10.7.3.8.4—Wave Equation Analysis

Add the following to the end of the article:

When the pile nominal resistance is greater than 600 kips or the pile diameter is greater than or equal to 18 inches, the wave equation analysis used for establishing the bearing acceptance criteria shall be based on dynamic test results with signal matching.

The wave equation shall be calibrated to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.4

Replace the 1st paragraph with the following:

Note that without dynamic test results with signal matching analysis and/or pile load test data (see Articles 10.7.3.8.2 and 10.7.3.8.3), some judgment is required to use the wave equation to predict the pile bearing resistance. Unless experience in similar soils exists, the recommendations of the software provider should be used for dynamic resistance input. Key soil input values that affect the predicted nominal resistance include the soil damping and quake values, the skin friction distribution, e.g., such as could be obtained from a static pile bearing analysis, and the anticipated amount of soil setup or relaxation. The actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though field observations such as hammer stroke or measured ram velocity can and should be used to improve the accuracy of the wave equation prediction. The reliability of the predicted pile axial nominal resistance can be improved by selecting the key input parameters based on local experience.

10.7.3.8.5—Dynamic Formula

Replace the 1st paragraph with the following:

If a dynamic formula is used to establish the driving criterion, the following modified Gates Formula (Eq. 10.7.3.8.5-1) shall be used. The nominal pile resistance as measured during driving using this method shall be taken as:

$$R_{ndr} = \left[1.83\sqrt{E_r}\,\log_{10}(0.83N_b)\right]\text{-}124 \quad (10.7.3.8.5\text{-}1)$$

where:

R_{ndr} = nominal pile resistance measured during pile driving (kips)

E_r = Manufacturer's rating for energy developed by the hammer at the observed field drop height (ft.-lbs.)

N_b = Number of hammer blows in the last foot, (maximum value to be used for N_b is 96) (blows/ft).

Delete the 2nd paragraph.

Delete the 3rd paragraph.

Replace the 5th paragraph with the following:

Dynamic formulas shall not be used when the required nominal resistance exceeds 600 kips or the pile diameter is greater than or equal to 18-inches.

C10.7.3.8.5

Delete the 2nd paragraph.

Delete the 3rd paragraph.

Replace the 5th paragraph with the following:

As the required nominal bearing resistance increases, the reliability of dynamic formulae tends to decrease. The modified Gates Formula tends to underpredict pile nominal resistance at higher resistances.

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10.7.3.8.6a—General

Add to the end of the article as follows:

The static analysis methods presented in this article shall be limited to driven piles 24 in. or less in diameter (length of side for square piles). For steel pipe and cast-in-steel shell (CISS) piles larger than 18 inches in diameter, the static analysis methods from the American Petroleum Institute (API, 2000) publication RP 2A shall be used.

C10.7.3.8.6a

Delete the entire commentary.

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10.7.3.10—Uplift Resistance of Single Piles

Replace the 1st paragraph with the following:

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads in uplift or tension.

Replace the 2nd paragraph with the following:

The uplift resistance of a single pile should be estimated in a manner similar to that for estimating the skin friction resistance of piles in compression specified in Article 10.7.3.8.6, and when appropriate, by considering reduction due to the effects of uplift.

C10.7.3.10

Add the following before the 1st paragraph as follows:

In general, piles may be considered to resist a transient, but not sustained, uplift load by skin friction.

Replace the 2nd paragraph with the following:

See Hannigan et al (2006) for guidance on the reduction of skin friction due to the effects of uplift.

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Replace the 5th paragraph of the article with the following:

The static pile uplift load test(s), when performed, should be used to calibrate the static analysis method, i.e., back calculate soil properties, to adjust the calculated uplift resistance for variations in the stratigraphy. The minimum penetration criterion to obtain the desired uplift resistance should be based on the calculated uplift resistance using the static pile uplift load test results, when available.

10.7.3.11—Uplift Resistance of Pile Groups

Replace the 4th paragraph with the following:

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of 1*H* in 4*V* from the base of the pile group taken from Figure 10.7.3.11-1. The nominal uplift resistance of the pile group when considered as a block shall be taken as equal to the weight of this soil block. Buoyant unit weights shall be used for soil below the groundwater level. In this case, the resistance factor φ_{ug} in Eq. 10.7.3.11-1 shall be taken as equal to 1.0.

Delete the 6th and 7th paragraphs.

C10.7.3.11

Add the following to the end of the commentary:

In cohesionless soils, the shear resistance around the perimeter of the soil block that will be uplifted is ignored. This results in a conservative estimate of the nominal uplift resistance of the block and justifies the use of a higher resistance factor of 1.0.

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10.7.3.13.1—Steel Piles

Add to the end of the article:

Shear rings are required in CISS piles and drilled shafts with permanent casing to ensure composite action.

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Replace the title of Article 10.7.5 with the following:

10.7.5—Protection Against Corrosion and Deterioration

Replace the 3rd paragraph with the following:

Soil, water, or site conditions that have a minimum resistivity equal to or less than 1100 ohm-cm shall be considered as indicators of potential pile corrosion or deterioration.

Delete the 4th paragraph.

Add the following after the 3rd paragraph:

A site is considered corrosive if one or more of the following soil, water, or site conditions exist:

- chloride concentration equal to or greater than 500ppm,
- sulfate concentration equal to or greater than 1500ppm,
- pH equal to or less than 5.5.

Steel piling may be used in corrosive soil and water environments provided that adequate corrosion mitigation measures are specified. When increased steel area is used for corrosion protection, the following corrosion rates shall be used to determine the corrosion allowance (sacrificial metal loss):

- 0.001 in. per year for soil embedment zone,
- 0.0015 in. per year for fill or disturbed natural soils,
- 0.002 in. per year for atmospheric zone (marine),
- 0.004 in. per year for immersed zone (marine),
- 0.006 in. per year for splash zone.

Designer must consider site specific corrosion rate for steel piling in scour zones.

The corrosion rates used to determine the corrosion allowance for steel piling shall be doubled for steel H-piling since there are two surfaces for the web and flanges that would be exposed to the corrosive environment.

C10.7.5

Replace the 9th paragraph with the following:

Deterioration of concrete piles can be reduced by design procedures. These include use of a dense impermeable concrete, sulfate resisting Portland cement, increased steel cover, air-entrainment, reduced chloride content in the concrete mix, cathodic protection, and epoxy-coated reinforcement. Piles that are continuously submerged are less subject to deterioration.

Delete the 10th paragraph.

10.8.1.1—Scope

C10.8.1.1

Add the following after the 2nd paragraph:

When casing is used to stabilize the soil within the excavation for construction of a Cast-In-Drilled Hole (CIDH) concrete pile, the method of installation will influence how the pile is designed and the resulting side and tip resistance. Special consideration shall be given to cases where oscillator or rotator drill equipment is used to construct CIDH concrete piles. Steel pipe (sometimes referred to as "casing") advanced into the ground using oscillator or rotator drilling equipment in most cases should be considered equivalent to large drilling rod with drilling teeth at the tip of the steel "casing." The drill teeth typically extend out slightly beyond the diameter of the oscillator or rotator drill rod resulting in a drilled hole larger than the outside diameter of the drill rod that is not "tight" in the hole. When oscillators and rotators are used to excavate earth materials, cuttings are produced outside of the drill rod (aka "casing") that are not typically removed during the drilling process or during the drill rod removal process, which can result in cuttings be trapped between the sidewalls of the excavations and the concrete of the pile.

In situations, where relatively large side resistance is relied upon in the design of CIDH concrete pile (e.g., IGM or rock), the contract specifications need to provide adequate requirements to ensure that the side resistance in the rock is not significantly reduced due to the use of oscillator/rotator drilling equipment to construct the pile. Some examples would include: 1) prohibiting the use of the oscillator/rotator drill rod in the rock socket portion of a CIDH concrete pile, or 2) after reaching the pile tip elevation, pull the rotator/oscillator drill rod up to the top of the rock and remove cuttings from the sidewalls of the excavation by other means prior to constructing the pile.

Studies have shown cases where significant reduction in side resistance of a CIDH concrete pile socketed in rock when an oscillator/rotator drill rod was used to construct the pile. The significant reduction in side resistance was presumably due to cuttings trapped between the concrete and rock along the sidewall of the excavation due to the method of installation. For further discussion regarding this topic, refer to Section 6 of the *Drilled Shafts: Construction Procedures and LRFD Design Methods* (Brown et al 2010) or the article titled "Deep Foundation Challenges At The New Benicia-Martinez Bridge" by the American Society of Civil Engineers (2004).

There have been a number of situations on projects in California where large ground subsidences have developed at the ground surface that was supporting the oscillator/rotator drilling equipment as a result of the means and methods used by the drilling contractor. To help prevent these situations, it is recommended that construction specifications be included in the contract documents to address these issues and the pile placement plans contain specific measure to avoid these issues. These include but are not limited to: 1) maintaining an adequate positive fluid head in wet excavations, 2) using only the approved slurry in wet excavations, 3) maintaining a soil plug (i.e. 10 ft) at the tip of the oscillator/rotator drill rod during excavation of the pile, and 4) specifying that contractors provide access to the top of the oscillator/rotator drill rod (i.e boom lift), so that inspectors can inspect the fluid head and monitor the progress of the excavation.

10.8.1.2—Shaft Spacing, Clearance, and Embedment Into Cap

Replace the 1st paragraph with the following:

The center-to-center spacing of drilled shafts in a group shall be not less than 2.5 times the shaft diameter. If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the sequence of construction shall be specified in the contract documents.

C10.8.1.2

Delete the commentary.

Add after the first paragraph of the article:

For abutments without a pile cap, the distance from the side of the shaft to the nearest edge of the abutment shall not be less than 6.0 inches.

Replace the article and title for Article 10.8.1.3 with the following:

10.8.1.3—Shaft Diameter, Concrete Cover, Rebar Spacing, and Enlarged Bases

If the shaft is to be manually inspected, the shaft diameter should not be less than 30.0 in. The diameter of columns supported by shafts should be smaller than or equal to the diameter of the drilled shaft. In order to facilitate construction of the drilled shafts (CIDH Piles), the minimum concrete cover to reinforcement shall be as specified in Table 10.8.1.3-1. For shaft capacity calculations, only 3” of cover is assumed effective and shall be used in calculations.

Table 10.8.1.3-1—Minimum Concrete Cover for Drilled Shafts (CIDH Piles)

Diameter of the Drilled Shaft (CIDH Pile) “D”	Side Concrete Cover
16” and 24” Standard Plan Piles	Refer to the applicable Standard Plans
24” ≤ D ≤ 36”	3”
42” ≤ D ≤ 54”	4”
60” ≤ D < 96”	5”
96” and larger	6”

In order to improve concrete flow when constructing drilled shafts, a minimum 5 in. x 5 in. clear window between the horizontal and vertical shaft reinforcing steel shall be maintained. The maximum center-to-center spacing of longitudinal bars in drilled shafts (CIDH Piles) is limited to 10 in. when the shaft diameter is less than 5 ft., and 12 in. for larger shafts.

At the locations of inspection pipes, 8.5 in of clear spacing shall be provided between the main longitudinal reinforcing bars adjacent to the inspection pipe, and the minimum clear spacing between the longitudinal reinforcing bars and the inspection pipe shall be 3.0 in.

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

C10.8.1.3

Replace the 3rd paragraph with the following:

In drilling rock sockets, it is common to use casing through the soil zone to temporarily support the soil to prevent cave-in, allow inspection and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. Depending on the method of excavation the diameter of the rock socket may need to be sized at least 8.0 in. smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

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10.8.2.2.2—Settlement of Single-Drilled Shaft

Add the following to the end of the article:

Superstructure tolerance to support movements shall be verified for the displacements assumed in the geotechnical design of the shaft at the strength limit states.

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10.8.3.5.1b—Side Resistance

Replace the 3rd paragraph with the following:

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed. Method of installation of a steel casing will dictate what design method shall be used in determining side resistance for the drilled shaft and casing portion of the pile. If a corrugated metal pipe is placed in an oversized hole and the annular space is properly backfilled with concrete or grout (e.g., tremie methods in wet conditions), use equation 10.8.3.5.1b-1 for estimating side resistance without reduction factors as long as concrete or grout placed in the annular space can be verified in the field to the satisfaction of the geotechnical designer. Smooth-wall steel casings installed by vibratory methods, oscillatory methods, rotational methods or placed in an excavated oversized hole shall not use equation 10.8.3.5.1b-1.

C10.8.3.5.1b

Replace the 3rd paragraph with the following:

Steel casing will generally reduce the side resistance of a cast-in-drilled hole concrete pile also known as a drilled shaft. No specific data is available regarding the reduction in skin friction of a drilled shaft in cohesive soil resulting from the use of permanent casing relative to concrete placed directly against the soil when the casing is vibrated, oscillated or rotated into the soil. Interface shear resistance for steel against cohesive soil can vary from 50 to 75 percent of the interface shear resistance for poured in place concrete against cohesive soil, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961).

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10.8.3.5.1c—Tip Resistance

Replace the 1st paragraph with the following:

For axially loaded shafts in cohesive soil, the net nominal unit tip resistance, q_p, in ksf, by the total stress method as provided in Brown et al (2010) shall be calculated as follows:

If $Z \geq 3D$,

$$q_p = N^*_c\, S_u \qquad (10.8.3.5.1c\text{-}1)$$

in which:

Table 10.8.3.5.1c-1—Bearing Capacity Factor N^*_c

Undrained shear strength, S_u (ksf)	N^*_c
0.5	6.5
1	8.0
2 - 5	9.0

Note: For S_u >5 to 50 ksf, use cohesive Intermediate Geomaterial procedures (Article 10.8.3.5.5).

If $Z \geq 3D$,

$$q_p = \left(\frac{2}{3}\right)\left[1+\left(\frac{1}{6}\right)\left(\frac{Z}{D}\right)\right] N^*_c\, S_u \qquad (10.8.3.5.1c\text{-}2)$$

where,

D = diameter of drilled shaft (ft)

Z = penetration length of drilled shaft in base cohesive layer (ft)

S_u = design undrained shear strength (ksf)

C10.8.3.5.1c

Delete the 2nd paragraph.

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10.8.3.5.2b—Side Resistance

Replace the article with the following:

The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as:

$$q_s = \beta\sigma'_v \leq 4.0 \quad \text{for} \quad 0.25 \leq \beta \leq 1.2 \qquad (10.8.3.5.2b\text{-}1)$$

in which, for sandy soils:

- $N_{60} \geq 15$:

$$\beta = 1.5 - 0.135\sqrt{z} \qquad (10.8.3.5.2b\text{-}2)$$

- $N_{60} < 15$:

$$\beta = \frac{N_{60}}{15}\left(1.5 - 0.135\sqrt{z}\right) \qquad (10.8.3.5.2b\text{-}3)$$

where:

σ'_v = vertical effective stress at soil layer mid-depth (ksf)
β = load transfer coefficient (dim)
z = depth below ground, at soil layer mid-depth (ft)
N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

Higher side resistance values may be used if verified by load tests.

For gravelly sands and gravels, Eq. 10.8.3.5.2b-4 should be used for computing β where $N_{60} \geq 15$. If $N_{60} < 15$, Eq. 10.8.3.5.2b-3 should be used.

$$B = 2.0 - 0.06(z)^{0.75} \qquad (10.8.3.5.2b\text{-}4)$$

When permanent casing is used, the method of installation of a steel casing will dictate what design method shall be used in determining side resistance for the cased portion of the drilled shaft. If a corrugated metal pipe is placed in an oversized hole and the annular space is properly backfilled with grout (e.g., tremie methods), use equation 10.8.3.5.2b-1 for estimating side resistance without reduction to the side resistance. Smooth-wall steel casings installed by vibratory methods, oscillatory methods, rotational methods or placed in an excavated oversized hole shall not use the equations in 10.8.3.5.2b.

C10.8.3.5.2b

Replace the 1st paragraph with the following:

O'Neill and Reese (1999) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 4.0 ksf in all soils.

O'Neill and Reese (1999) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the SPT blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.

The detailed development of Eq. 10.8.3.5.2b-4 is provided in O'Neill and Reese (1999).

The design method by Chen and Kulhawy (2002) provides an alternate approach to calculating side resistance for drilled shafts. The design method was shown to be very sensitive to soil type and allowed for a reduction of 2/3 to the horizontal stress coefficient when construction quality was not properly controlled. For these reasons, the Chen and Kulhawy (2002) design method can be considered for use only if verified with load tests.

Replace the 2nd paragraph with the following:

Steel casing will generally reduce the side resistance of a cast-in-drilled-hole concrete pile also known as a drilled shaft. No specific data is available regarding the reduction in skin friction for drilled shafts in cohesionless soil resulting from the use of permanent casing relative to concrete placed directly against the soil when the casing is vibrated, oscillated or rotated into the soil. Interface shear resistance for steel against cohesionless soil can vary from 50 to 75 percent of the interface shear resistance for poured in place concrete against cohesionless soil, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Note that unit side resistance for poured in place concrete against cohesionless soil is nearly equal to the soil shear strength in most cases.

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10.8.3.6.3—Cohesionless Soil

Replace Table 10.8.3.6.3-1 with the following:

Table 10.8.3.6.3-1—Group Reduction Factors for Bearing Resistance of Shafts in Sand

Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Reduction Factor for Group Effects, η
Single Row	2.5*D*		0.95
	3*D* or more		1.0
Multiple Row	2.5*D*		0.67
	3*D*		0.80
	4*D* or more		1.0
Single and Multiple Rows	2.5*D* or more	Shaft group cap in intimate contact with ground consisting of medium dense or denser soil, and no scour below the shaft cap is anticipated	1.0
Single and Multiple Rows	2.5*D* or more	Pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted.	1.0

10.8.3.7.2—Uplift Resistance of Single Drilled Shaft

Replace the 1st paragraph with the following:

The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in Article 10.8.3.5, and, when appropriate, by considering reduction due to effects of uplift.

C10.8.3.7.2

Replace the 1st paragraph with the following:

The side resistance for uplift is lower than that for axial compression. One reason for this is that drilled shafts in tension unload soils, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft.

10.8.3.9.3—Reinforcement

Replace 1st paragraph with the following:

Where the potential for lateral loading is insignificant, drilled shafts may be reinforced for axial load only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with article 5.6.4. For drilled shafts with a diameter larger than 24 inches, reinforcing steel shall extend 6 inches above the pile specified tip elevation. For *Standard Plan* CIDH piles, the cover to reinforcing steel shall be as shown on the plans.

10.8.3.9.4—Transverse Reinforcement

Add a new paragraph to the end of the article:

The design shear force demand in CIDH shafts and rock sockets need not be more than two and a half times the seismic overstrength shear force of the column: $V_u \leq 2.5V_o$

Add a new commentary:

C10.8.3.9.4

Caltrans policy imposes an upper limit on the design shear force, recognizing the general problem of unrealistic shear magnification due to abrupt changes in stiffness, and discretization of distributed soil reaction at nodal points in rock.

Replace the article and title for Article 10.9.1.2 with the following:

10.9.1.2—Maximum Micropile Diameter and Minimum Micropile Spacing, Clearance, and Embedment into Cap

Center-to-center spacing of micropiles shall not be less than 30.0 in. or 3.0 pile diameters, whichever is greater. Otherwise, the provisions of Article 10.7.1.2 shall apply. The diameter of the micropile drilled hole shall not be greater than 13 inches.

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10.9.3.5.4—Micropile Load Test

Delete the article in its entirety.

C10.9.3.5.4

Delete the article in its entirety.

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10.10—REFERENCES

Add the following reference:

Gu. R. X., et. al. 2004 "Deep Foundation Challenges At The New Benicia-Martinez Bridge." In *Geotechnical Engineering for Transportation Projects,* Geotechnical Special Publication No. 126. American Society of Civil Engineers. pp. 1183-1191.

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TABLE OF CONTENTS

Replace Article 11.6.5 of the Table of Contents with the following:

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11.3—NOTATION

Replace the notations with the following:

B = wall base width (ft); semi-gravity wall heel length (ft) (11.10.2) (A11.3.1)
b = gross width of the strip, sheet, or grid reinforcement; width of bin module (ft) (11.10.6.4.1) (11.11.5.1)
D_{min} = distance between the back of MSE facing elements and any concrete footing element (ft) (11.10.11)
H = height of wall (ft); vertical distance between ground surface and top of heel at the stem (ft) (11.6.5.1) (A11.3.1)
H_{max} = maximum clear distance between superstructure soffit and finished grade in front of the MSE facing (ft) (11.10.11)

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11.5.1—General

Replace the 2nd paragraph with the following:

Abutments, piers and retaining walls shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads (if applicable) in accordance with the general principles specified in this Section.

11.5.2—Eccentricity Limits

Add a new paragraph after the 2nd paragraph:

When abutments and piers are supported on shallow foundations, the location of the resultant of the reaction forces shall be in compliance with the provisions of Article 10.5.2.2.

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Add a new paragraph after the last paragraph:

The eccentricity check for bridge shallow foundations, including abutments, is conducted under Service-I Combination as stated in Section 10.

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11.5.6—Load Combinations and Load Factors

Replace the 1st paragraph with the following:

Piers and retaining structures and their foundations and other supporting elements shall be proportioned for all applicable load combinations specified in Article 3.4.1. Abutments and their foundations shall be proportioned for all applicable load combinations specified in Article 3.4.5.

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11.5.7—Resistance Factors – Service and Strength

Replace Table 11.5.7-1 with the following:

Table 11.5.7-1—Resistance Factors for Permanent Retaining Walls

Wall-Type and Condition		Resistance Factor
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		1.00
Pullout resistance of anchors (1)	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 (1) 0.70 (1) 0.50 (1)
Pullout resistance of anchors (2)	• Where proof tests are conducted	1.0 (2)
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A 615 bars) • High strength steel (e.g., ASTM A 722 bars) • High strength steel strands (e.g., ASTM A 416)	0.90 (3) 0.80 (3) 0.75 (3)
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semigravity Walls		
Bearing resistance	• Gravity and semi-gravity walls • MSE walls	0.55 0.65
Sliding	• Friction • Passive resistance	1.00 0.50
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements (4) • Static loading Grid reinforcements (4) (5) • Static loading	0.90 0.80
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

Continued on next page

Table 11.5.7-1 (Continued)—Resistance Factors for Permanent Retaining Walls

(1) Apply to presumptive ultimate unit bond stresses for preliminary design only in Article C11.9.4.2.

(2) Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.

(3) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(4) Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

(5) Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

11.6.1.2—Loading

Replace the 2nd paragraph with the following:

The provisions of Articles 3.11.5 and 11.5.5 shall apply. For stability computations, the earth loads shall be multiplied by the maximum and/or minimum load factors given in Table 3.4.1-2, as appropriate. Abutments and their foundations shall be proportioned for all applicable load combinations specified in Article 3.4.5.

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11.6.1.5.2—Wingwalls

Replace the article with the following:

Reinforcing bars or suitable rolled sections shall be spaced across the junction between wingwalls and abutments to tie them together. Such bars shall extend into the concrete and/or masonry on each side of the joint far enough to develop the strength of the bar as specified for bar reinforcement, and shall vary in length so as to avoid planes of weakness in the concrete at their ends. If bars are not used, an expansion joint shall be provided and the wingwall shall be keyed into the body of the abutment.

Replace the article and title for Article 11.6.1.6 with the following:

11.6.1.6—Expansion and Weakened Plane Joints

Weakened plane joints shall be provided at intervals not exceeding 24.0 ft and expansion joints at intervals not exceeding 96.0 ft for conventional retaining walls. All joints shall be filled with approved filling material to ensure the function of the joint.

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11.6.3.3—Eccentricity Limits

Replace the article with the following:

For shallow foundations of retaining walls on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

The eccentricity check for bridge shallow foundations, including abutments, is conducted under Service-I Combination as stated in Section 10.

For shallow foundations of retaining walls on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths of the base width.

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Replace the article title for 11.6.5 with the following:

11.6.5—Seismic Design for Conventional Retaining Walls

Add a new commentary as follows:

C11.6.5

Abutments founded in Class S1 soil (as defined in SDC 6.1.2) have been exempted from Extreme Event (Seismic) design considering the following facts:

- Post seismic observations have not shown any catastrophic damage to abutments that resulted in collapse, provided that enough seat width has been provided for superstructure movements.
- For non-integral type abutments, excessive movement of the abutment towards the bridge is prevented by contact of the back wall to the superstructure.
- Components of the abutments, such as shear keys and the backwall, are designed to break without causing any failure in the foundation system.
- Overall (slope) stability check is performed by the geotechnical professional.

Abutments in Class S2 soil (as defined in SDC 6.1.3) require special analysis.

11.6.5.1—General

Replace the 1st sentence of the 1st paragraph with the following:

Rigid gravity and semigravity retaining walls shall be designed to meet overall stability, external stability, and internal stability requirements during seismic loading.

Delete the 3rd paragraph.

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Replace the article and title for Article 11.6.5.4 with the following:

11.6.5.4—Calculation of Seismic Earth Pressure for Nonyielding Walls

For walls that are considered nonyielding, the value k_h used to calculate seismic earth pressure shall be increased to 1.0k_{h0}, unless the Owner approves the use of more sophisticated numerical analysis techniques to determine the seismically induced earth pressure acting on the wall to yield in response to lateral loading. In this case, k_h should not be corrected for wall displacement, since displacement is assumed to be zero. However, k_h should be corrected for wave scattering effects as specified in Article 11.6.5.2.2.

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11.9.5.1—Anchors

C11.9.5.1

Add the following after the 2nd paragraph:

In addition to the Hinge Method shown in Figures C11.9.5.1-1 and C11.9.5.1-2, the Modified Hinge Method may be used as illustrated in Figure C11.5.9.1-3 a) for one-level anchor and C11.9.5.1-3 b) for multilevel anchors. The Modified Hinge Method eliminates the reaction *"R"* at the base of the excavation by applying earth pressures to the vertical wall element below the base of excavation. The pressure on the vertical wall element in Figure C11.9.5.1-3 is shown schematically and for illustrative purposes only. The actual active and passive pressure diagrams on the vertical wall element will depend on soil parameters unique to the wall system and on appropriate earth pressure theories.

The procedure to analyze a one-level wall in Figure C11.9.5.1-3a) is as follows:

i) Take moments about the anchor to calculate the embedment depth of the vertical wall element, *CD*;
ii) Set summation of forces equal to zero in the horizontal direction to calculate anchor load T_1;
iii) Calculate Maximum Bending Moment (M_{MAX}) and Maximum Shear Force (V_{MAX}) in the wall element.

The procedure to analyze a multilevel wall in Figure C11.9.5.1-3b is,

i) Calculate T_{NU} in the lowest anchor following the procedure given for the Hinge Method, using the loads in the anchors above the lowest anchor;
ii) Calculate M_{MAX} and V_{MAX} in the wall element *ACD*;
iii) Calculate the embedment depth of the vertical wall element, *EL*, by taking moments about the lowest anchor;
iv) Set summation of forces equal to zero in the horizontal direction for *DEL* to calculate the lowest anchor load (T_{nL});
v) The total load in the lowest anchor, T_n, is T_{nU} + T_{nL};
vi) Calculate M_{MAX} and V_{MAX} in the wall element, *DEL*;
vii) Design the pile with the controlling M_{MAX} and V_{MAX} from *ACD* and *DEL*.

Add the following figure after Figure C11.9.5.1-2:

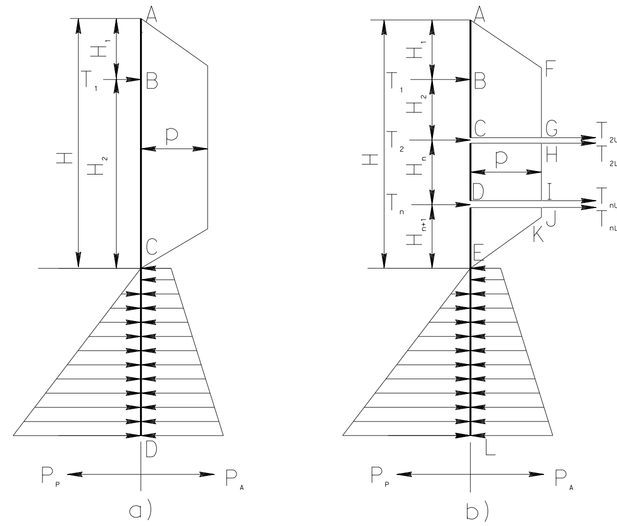

Figure C11.9.5.1-3—Calculation of Anchor Loads and Pile Embedment Depth for a) One-Level Wall and b) Multilevel Wall

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11.10.2.2—Minimum Front Face Embedment

Replace the 2nd bullet of the 2nd paragraph with the following:

- 10% of the design height, and not less than 2.0 ft

Replace the 5th paragraph with the following:

A minimum horizontal bench width of 0.1H but not less than 4.0 ft shall be provided in front of walls founded on slopes. The bench may be formed or the slope continued above that level as shown in Figure 11.10.2-1.

C11.10.2.2

Delete Table C11.10.2.2-1

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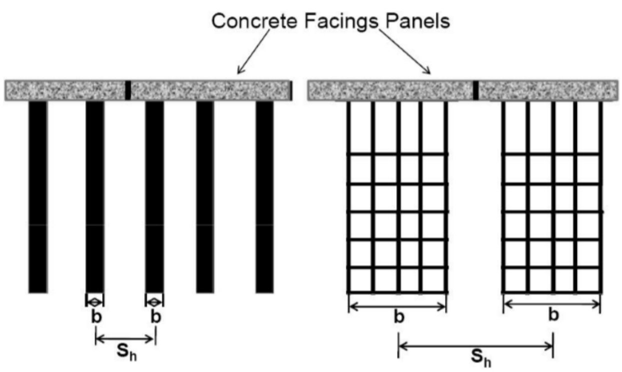
11.10.6.4.1—General

Replace the definition of *b* within Figure 11.10.6.4.1-1 with the following:

b = the gross width of the strip, sheet, or grid reinforcement (if reinforcement is continuous, count the number of bars for reinforcement width of 1 unit of measure).

Add a new figure after Figure 11.10.6.4.1-1 as follows:



b = the gross width of the strip, sheet, or grid reinforcement.

Figure 11.10.6.4.1-1a—Reinforcement Width *b*

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11.10.6.4.2a— Steel Reinforcements

Add a new paragraph before the 4th paragraph:

When soil backfill conforms to the following criteria:

- pH = 5.5 to 10
- Resistivity ≥2000 ohm-cm
- Chlorides <250 ppm
- Sulfates <500 ppm
- Organic Content ≤1 percent
- Does not contain slag aggregate or recycled materials

the sacrificial thicknesses shall be computed for each exposed surface as follows:

- Loss of galvanizing over 10 years (using 2 oz/ ft^2)
- Loss of carbon steel = 1.1 mil/yr. after zinc depletion

C11.10.6.4.2a

Add a new paragraph after the 4th paragraph:

Considerable data from numerous MSE in California has been gathered for a national research project to develop the resistance and load factors for corrosion in actual field conditions. As a result, the equations, design parameters and construction specifications are under review. This section continues current practice in conjunction with the more aggressive soils permitted in current Caltrans construction specifications, until that review is complete.

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11.10.11—MSE Abutments

Replace the 6th paragraph with the following:

The minimum thickness of compacted backfill between the concrete footing elements and the soil reinforcement shall be 6 inches. The minimum distance, D_{min}, between the back of the MSE facing elements and any element of the concrete footing shall be as follows:

$$D_{min} = 8 - 0.3(20 - H_{max}) \geq 5 \text{ ft} \tag{11.10.11-3}$$

where H_{max} is ≤ 30 ft

Replace the 9th paragraph with the following:

In pile or drilled shaft supported abutments, the horizontal forces transmitted to the deep foundation elements shall be resisted by the lateral capacity of the deep foundation elements by provision of additional reinforcements to tie the drilled shaft or pile cap into the soil mass, or by batter piles. Lateral loads transmitted from the deep foundation elements to the reinforced backfill may be determined using a P-Y lateral load analysis technique. The facing shall be isolated from horizontal loads associated with lateral pile or drilled shaft deflections. A minimum clear distance of 5.0 ft shall be provided between the facing and all deep foundation elements. Piles or drilled shafts shall be specified to be placed prior to wall construction and cased through the fill if necessary.

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A11.3.1—Mononobe-Okabe Method

Replace Figure A11.3.1-1 with the following:

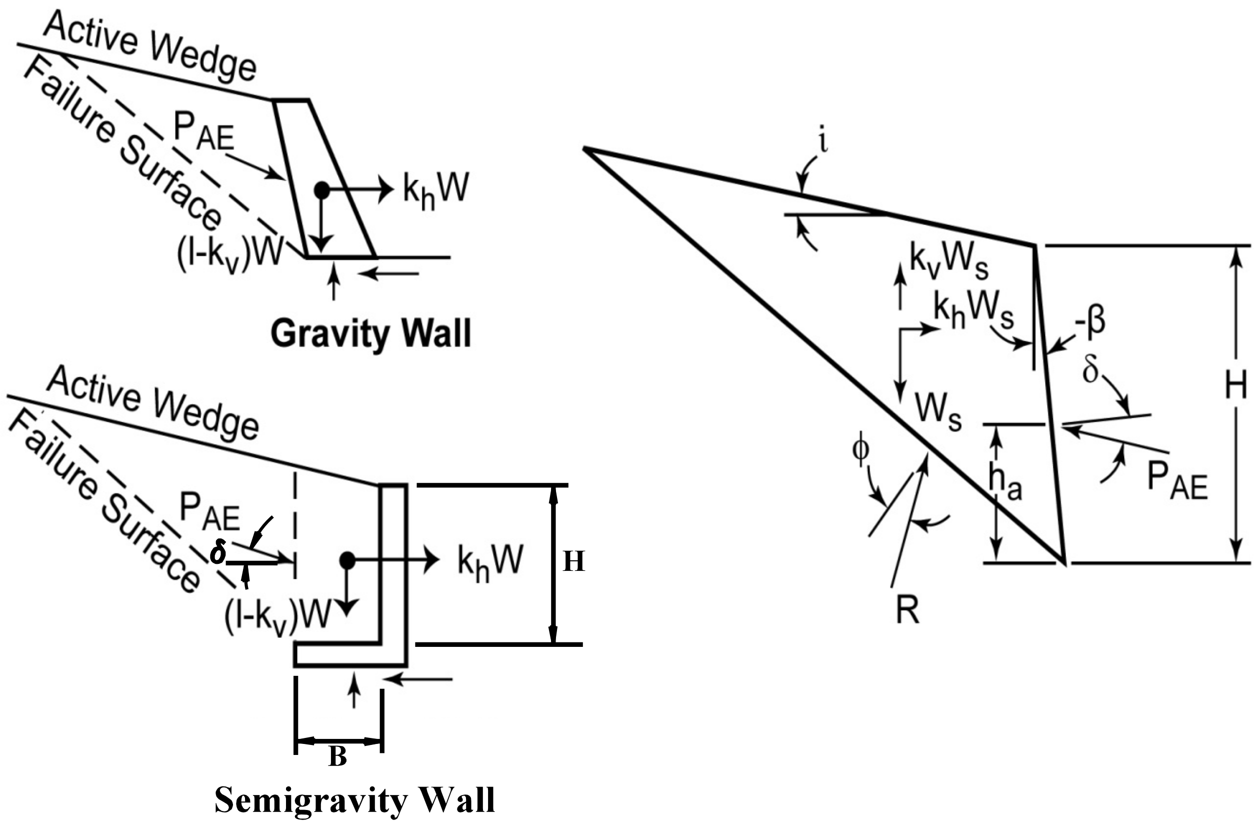

Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

Add the following before the second to last paragraph:

Modified from a study by Iskander, *et al* (2013), when retaining cohesionless soil, the angle of P_{AE} on the vertical plane through the end of the heel of a semigravity wall, δ, in Figure A11.3.1-1 may be determined as,

$$\delta = \tan^{-1}\left\{\frac{\sin(2\theta_{Mo}) + m_{\alpha}\sin(2i)}{2\,[\sin^2\theta_{Mo} + m_{\alpha}\cos^2 i\,]}\right\}\text{(degrees)} \qquad \text{(A11.3.1-2)}$$

where

$$m_{\alpha} = \frac{\cos(i+\theta_{Mo}) - \sqrt{\cos^2(i+\theta_{Mo}) - \cos^2\phi_f}}{\cos(i+\theta_{Mo}) + \sqrt{\cos^2(i+\theta_{Mo}) - \cos^2\phi_f}} \qquad \text{(A11.3.1-3)}$$

δ used with equation (A11.3.1-2) is measured from horizontal. Modified from a study by Kloukinas, *et al* (2011), the heel of the semigravity wall must be long enough for δ in Equation A11.3.1-2 to be valid and the ratio of heel length, *B*, over stem height, *H*, must satisfy the following relation,

$$\frac{B}{H} \geq \tan\left(45° - \frac{\phi_f}{2} - \frac{\Delta_{1e} - i}{2} - \frac{\theta_{Mo}}{2}\right) \tag{A11.3.1-4}$$

where

$$\Delta_{1e} = \sin^{-1}\left\{\frac{\sin(i + \theta_{Mo})}{\sin\phi_f}\right\} \text{(degrees)} \tag{A11.3.1-5}$$

Equation A11.3.1-2 should not be used to predict δ, for any other types of earth retaining systems, including, but not limited to, gravity walls, MSEs, non-gravity walls, crib walls or gabion systems.

12.6.6.1—Trench Installations

Replace the 1st paragraph with the following:

The minimum trench width shall provide a 24-in. minimum side wall clearance between the pipe and the trench wall to ensure sufficient working room to properly and safely place and compact backfill material.

C12.6.6.1

Replace the 1st paragraph with the following:

The use of specially designed equipment may enable satisfactory installation and embedment even in narrower trenches. If the use of such equipment provides an installation meeting the requirements of this Article, narrower trench widths may be used as approved by the Engineer.

Replace the 2nd paragraph with the following:

For trenches excavated in rock or high-bearing soils, decreased trench widths may be used up to the limits required for compaction. For these conditions, the use of a flowable backfill material, as specified in Article 12.4.1.3, allows the envelope to be decreased to within 6.0 in. along each side of the pipe for pipes up to and including 42 inches in diameter or span, or 12 inches for pipes over 42 inches in diameter or span.

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12.6.6.2—Embankment Installations

C12.6.6.2

Replace Table C12.6.6.2-1 with the following:

Table C12.6.6.2-1—Minimum Width of Soil Envelope

Diameter, *S* (in.)	Minimum Envelope Width (ft)
<24	2.0
24–108	2.0
>108	5.0

12.6.6.3—Minimum Cover

Replace Table 12.6.6.3-1 with the following:

Table 12.6.6.3-1—Minimum Cover

Type	Condition	Minimum Cover*
Corrugated Metal Pipe		$S/8 \geq 24.0$ in.
Spiral Rib Metal Pipe	Steel Conduit	$S/4 \geq 24.0$ in.
	Aluminum Conduit where $S \leq 48.0$ in.	$S/2 \geq 24.0$ in.
	Aluminum Conduit where $S > 48.0$ in.	$S/2.75 \geq 24.0$ in.
Structural Plate Pipe Structures		$S/8 \geq 24.0$ in.
Long-Span Structural Plate Pipe Structures		Refer to Table 12.8.3.1.1-1
Structural Plate Box Structures		1.4 ft as specified in Article 12.9.1
Deep Corrugated Structural Plate Structures		See Article 12.8.9.4
Fiberglass Pipe		24.0 in.
Thermoplastic Pipe	Under unpaved areas	$ID/8 \geq 24.0$ in.
	Under paved roads	$ID/2 \geq 24.0$ in.
Steel-Reinforced Thermoplastic Culverts		$S/5 \geq 24.0$ in.
* Minimum cover taken from top of rigid pavement or bottom of flexible pavement		
Reinforced Concrete Pipe	Under unpaved areas or top of flexible pavement	$B_c/8$ or $B_c/8$ whichever is greater, ≥ 24.0 in.
Reinforced Concrete Pipe	Under bottom of rigid pavement	12.0 in.

12.10.2.1—Standard Installations

Replace Table 12.10.2.1-1 with the following:

Table 12.10.2.1-1—Standard Embankment Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, use B_c/2.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	95% SW	90% SW, 95% ML, or 100% CL
Type 2—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use B_c/2.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	90% SW or 95% ML	85% SW, 90% ML, or 95% CL
Type 3—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use B_c/2.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL

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12.10.2.1—Standard Installations

Replace Table 12.10.2.1-2 with the following:

Table 12.10.2.1-2—Standard Trench Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, use B_c/2.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	95% SW	90% SW, 95% ML, or 100% CL, or natural soils of equal firmness
Type 2—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use B_c/2.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	90% SW or 95% ML	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 3—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use B_c/4.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c in. minimum, not less than 6.0 in.	85% SW, 90% ML or 95% CL	85% SW, 90% ML, 95% CL, or natural soils of equal firmness

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12.10.2.1—Standard Installations

Replace Table 12.10.2.1-3 with the following:

Table 12.10.2.1-3—Coefficients for Use with Figure 12.10.2.1-1

	Installation Type		
	1	2	3
VAF	1.35	1.40	1.40
HAF	0.45	0.40	0.37
A1	0.62	0.85	1.05
A2	0.73	0.55	0.35
A3	1.35	1.40	1.40
A4	0.19	0.15	0.10
iA5	0.08	0.08	0.10
A6	0.18	0.17	0.17
a	1.40	1.45	1.45
b	0.40	0.40	0.36
c	0.18	0.19	0.20
e	0.08	0.10	0.12
f	0.05	0.05	0.05
u	0.80	0.82	0.85
v	0.80	0.70	0.60

12.10.2.3—Live Loads

Add the following to the end of the article:

The unfactored live load W_L shall be determined as:

$$W_L = P_L \times C_L \times B_C \qquad (12.10.2.3\text{-}1)$$

where:

W_L = live load on pipe (kip/ft)
P_L = live load pressure as defined in Eq. 3.6.1.2.6b-7 (ksf)

$$C_L = \frac{L_w}{B_c} \leq 1.0$$

L_w = live load distribution length in the circumferential direction as specified in Article 3.6.1.2.6 (ft)

Add an additional article with a paragraph and three figures:

12.10.2.4—Non-Standard Installations

When non-standard installations are used, the unfactored earth pressure on the structure shall be the prism of earth weight (prism load) above the pipe multiplied by a soil-structure interaction factor. The unit weight of soil shall not be less than 120 lb/cu. ft. In the case that a more accurate estimate of the unit weight of soil is required, the maximum unit weight can be verified through a lab test by geotechnical engineers. Pressure distribution shall be determined by an appropriate soil-structure interaction analysis. Acceptable pressure distributions for non-standard installations are: the Olander/Modified Olander Radial Pressure Distribution - see Figure 12.10.2.4-1(a), or the Paris/Manual Uniform Pressure Distribution - see Figure 12.10.2.4-1(b). For bedding angles and lateral pressures used with the latter distributions see Figure 12.10.2.4-2 and Figure 12.10.2.4-3. Other methods for determining total load and pressure distribution may be used, if based on successful design practice or tests that reflect the appropriate design condition.

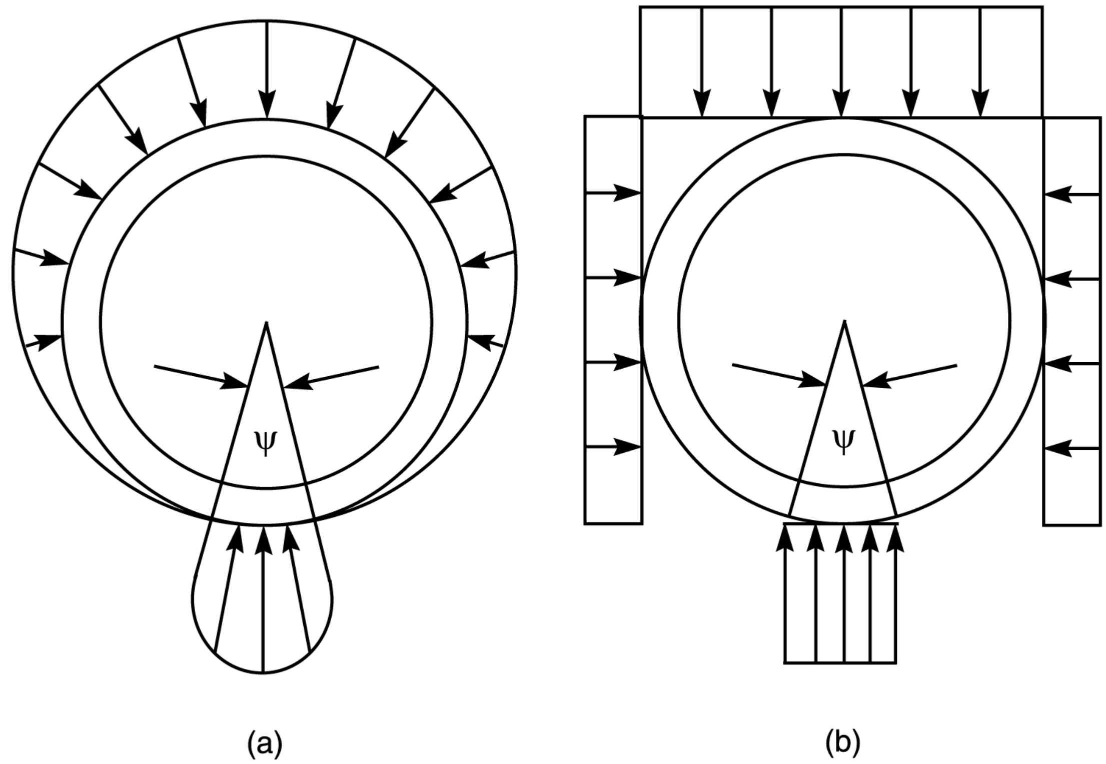


Figure 12.10.2.4-1(a)—Olander/Modified Olander Radial Pressure Distribution Diagram
Figure 12.10.2.4-1(b)—Paris/Manual Uniform Pressure Distribution Diagram

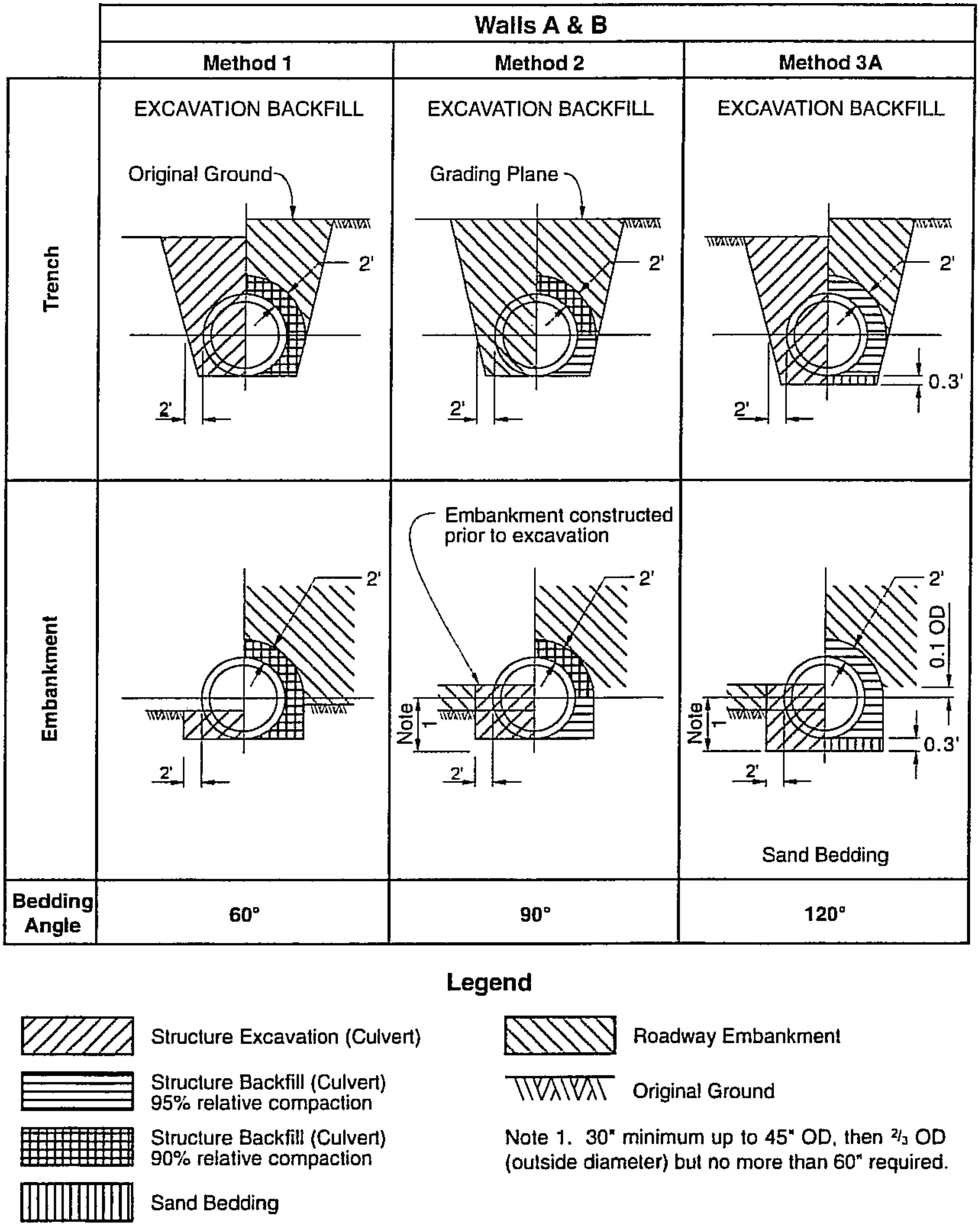
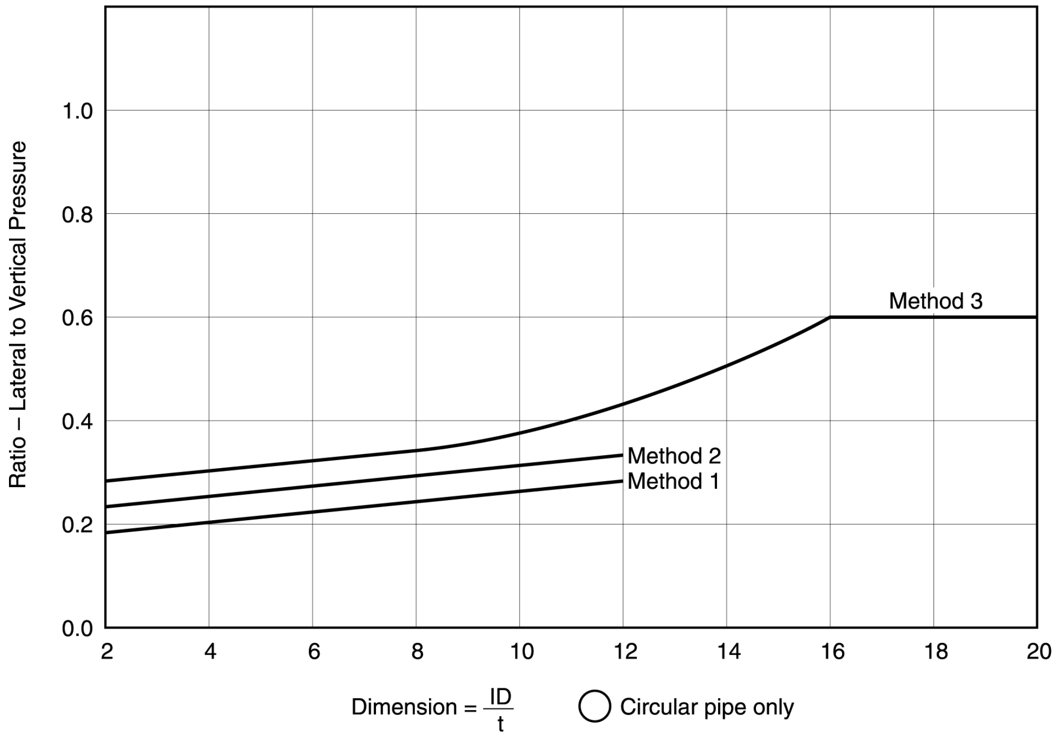


Figure 12.10.2.4-2—Trench and Embankment Backfill Bedding Angles



LATERAL PRESSURE

Legend
ID = inside diameter of pipe, t = wall thickness of pipe

Figure 12.10.2.4-3—Non-Standard Installation Lateral Pressures Distribution

12.10.4.3—Indirect Design Method

12.10.4.3.1—Bearing Resistance

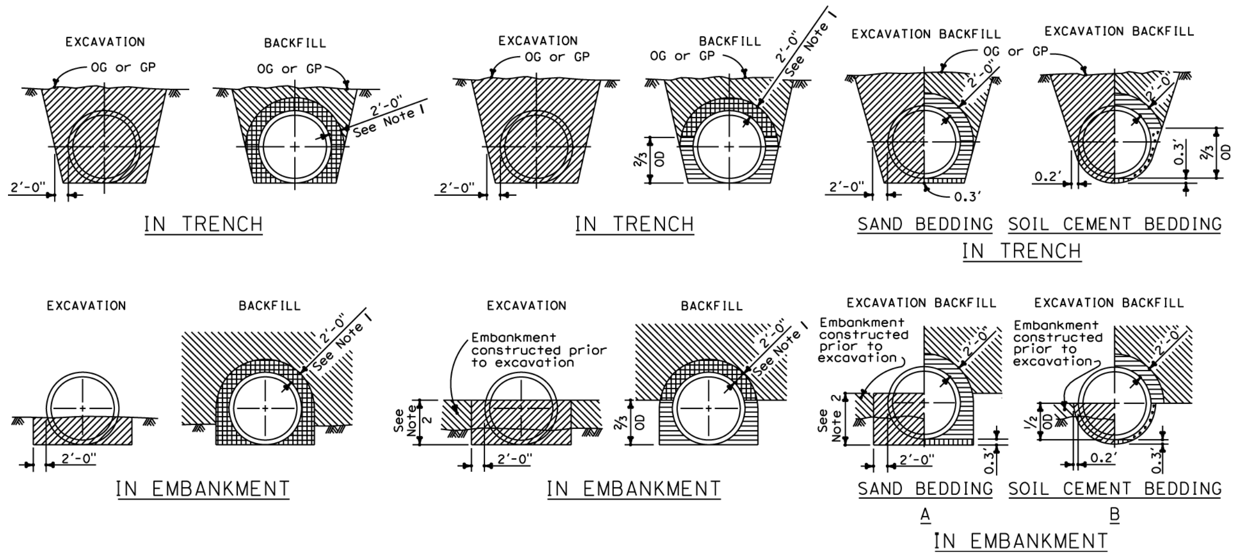
Add a new 2nd paragraph, a figure and a table after the 1st paragraph as follows:

Reinforced concrete pipe culvert excavation/backfill criteria for Caltrans non-standard installation Methods 1, 2, and 3 are summarized in Figure 12.10.4.3.1-1 below. Associated fill heights and pipe classes are indicated in the adjacent D-Load Overfill Table 12.10.4.3.1-1. Pipe backfill is to be placed over the full width of excavation except where dimensions are shown for specific backfill width or thickness. Dimensions shown are minimums.

C12.10.4.3.1—Bearing Resistance

Add a new paragraph to the end of the commentary as follows:

Above information is based on Caltrans research (*Transportation Record* 878) and Caltrans *Standard Plans* 2015 A62D



METHOD 1 **METHOD 2** **METHOD 3**

Notes:

1. Embankment compaction requirements govern over the 90% relative compaction backfill requirement within 2′-6″ of finished grade.
2. Embankment height prior to excavation for installation of all classes of RCP under Method 2 and Method 3A shall be as follows:

   Pipe sizes 1′-0″ to 3′-6″ H = 2′-6″
   Pipe sizes 4′-0″ to 7′-0″ H = 2/3 OD
   Pipe sizes larger than 7′-0″ H = 5′-0″

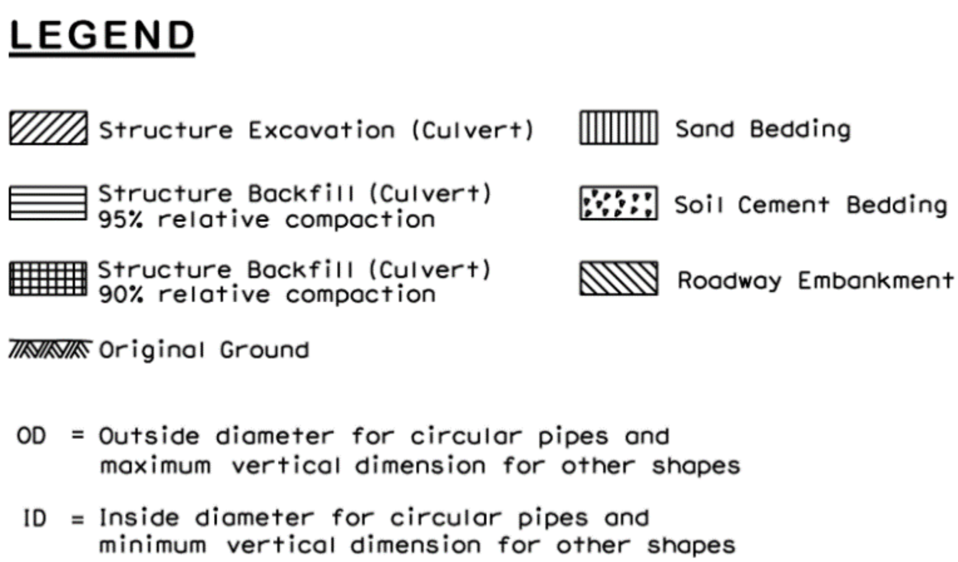


Figure 12.10.4.3.1-1—Non-Standard Installation Excavation and Backfill

Table 12.10.4.3.1-1—D-Load Overfill Table

MINIMUM ALLOWABLE CLASSES OF RCP FOR METHOD 1

COVER	MINIMUM CLASS AND D-LOAD	
5.9′	Class II	1000D
6.0′ - 7.9′	Class III	1350D
8.0′ - 9.9′	Class III	Special 1700D
10.0′ - 11.9′	Class IV	2000D
12.0′ - 13.9′	Class IV	Special 2500D
14.0′ - 16.9′	Class V	3000D
17.0′ - 20.0′	Class V	Special 3600D

METHOD 1

MINIMUM ALLOWABLE CLASSES OF RCP FOR METHOD 2

COVER	MINIMUM CLASS AND D-LOAD	
15.9′	Class II	1000D
16.0′ - 19.9′	Class III	1350D
20.0′ - 24.9′	Class III	Special 1700D
25.0′ - 27.9′	Class IV	2000D
28.0′ - 34.9′	Class IV	Special 2500D
35.0′ - 41.9′	Class V	3000D
42.0′ - 50.0′	Class V	Special 3600D

METHOD 2

MINIMUM ALLOWABLE CLASSES OF RCP FOR METHOD 3

COVER	MINIMUM CLASS AND D-LOAD	
25.9′	Class II	1000D
26.0′ - 31.9′	Class III	1350D
32.0′ - 37.9′	Class III	Special 1700D
38.0′ - 44.9′	Class IV	2000D
45.0′ - 55.9′	Class IV	Special 2500D
56.0′ - 67.9′	Class V	3000D
68.0′ - 80.0′	Class V	Special 3600D

METHOD 3

REINFORCED CONCRETE PIPE

Note: The maximum size for all classes or RCP placed under Method 1 is 78″ ID.

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12.10.4.3.2a—Earth Load Bedding Factor for Circular Pipe

Replace Table 12.10.4.3.2a-1 with the following:

Table 12.10.4.3.2a-1—Bedding Factors for Circular Pipe

Pipe Diameter, in.	Standard Installations		
	Type 1	Type 2	Type 3
12	4.4	3.2	2.5
24	4.2	3.0	2.4
36	4.0	2.9	2.3
72	3.8	2.8	2.2
144	3.6	2.8	2.2

12.10.4.3.2c—Live Load Bedding Factors

Replace the entire article with the following:

The bedding factor B_{FLL} for live load, W_L, for circular, arch, and elliptical pipe shall be taken as specified in Table 12.10.4.3.2c-1. For pipe diameters not listed in Table 12.10.4.3.2c-1, the bedding factor may be determined by interpolation.

C12.10.4.3.2c

Replace the commentary with the following:

When the live load becomes essentially uniform across the top of the pipe, the basic live load bedding factor is 2.2. For larger pipe this occurs at a greater depth. For shallow depths the live load will be concentrated over only a small portion of a large diameter pipe, thus resulting in a higher moment (lower bedding factor) for the same total load.

Table 12.10.4.3.2c-1—Bedding Factors, B_{FLL}

Fill Height, ft	Pipe Diameter, in.										
	12	24	36	48	60	72	84	96	108	120	144
0.5	2.2	1.7	1.4	1.3	1.3	1.1	1.1	1.1	1.1	1.1	1.1
1.0	2.2	2.2	1.7	1.5	1.4	1.3	1.3	1.3	1.1	1.1	1.1
1.5	2.2	2.2	2.1	1.8	1.5	1.4	1.4	1.3	1.3	1.3	1.1
2.0	2.2	2.2	2.2	2.0	1.8	1.5	1.5	1.4	1.4	1.3	1.3
2.5	2.2	2.2	2.2	2.2	2.0	1.8	1.7	1.5	1.4	1.4	1.3
3.0	2.2	2.2	2.2	2.2	2.2	2.2	1.8	1.7	1.5	1.5	1.4
3.5	2.2	2.2	2.2	2.2	2.2	2.2	1.9	1.8	1.7	1.5	1.4
4.0	2.2	2.2	2.2	2.2	2.2	2.2	2.1	1.9	1.8	1.7	1.5
4.5	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.0	1.9	1.8	1.7
5.0	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.0	1.9	1.8
5.5	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.0	1.9
6.0	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.1	2.0
6.5	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2

13.9.2—Geometry

Replace the article with the following:

The height of a bicycle railing shall not be less than 42.0 in., measured from the top of the riding surface. If the bicycle railing and the vehicular rail were not successfully crash tested as an integral unit, the bicycle railing shall be offset a minimum of 15.0 in. behind the face of the vehicular rail.

The height of an in-plane railing for bicycles only shall not be less than 48.0 in. measured from the top of the riding surface.

Bicycle railings shall have rail spacing satisfying the respective provisions of Article 13.8.1.

If deemed necessary, rubrails attached to the rail or fence to prevent snagging should be deep enough to protect a wide range of bicycle handlebar heights.

If screening, fencing, or a solid face is utilized, the number of rails may be reduced.

C13.9.2

Replace the commentary with the following:

Railings, fences or barriers on either side of a shared use path on a structure, or along bicycle lane, shared use path or signed shared roadway located on a highway bridge should be a minimum of 42.0 in. high. The 42.0 in. minimum height is in accordance with the AASHTO Guide for the Development of Bicycle Facilities, Third Edition (1999).

The 15-inch bicycle rail offset behind the face of the vehicular rail is required to maintain the vehicular crash test certification if the vehicular rail and bicycle railing were not crash tested as an integral unit.

In-plane bicycle railing refers to bicycle railing that is:

- not working in combination with vehicular rail, such as along a bikepath where bicycle traffic is separated from vehicular traffic, and
- in-plane for the full height with no offset in the upper portion.

On such a bridge or bridge approach where high speed high angle impact with railing, fence or barrier are more likely to occur (such as short-radius curves with restricted site distance or at the end of a long grade) or in locations with site specific safety concerns, a railing, fence or barrier height above the minimum should be considered.

The need for rubrails attached to a rail or fence is controversial among many bicyclists.

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A13.4.1—Design Cases

Add the following after the last paragraph:

Design Case 1 and Design Case 2 are also applicable for the evaluation of existing deck overhangs supporting new railings.

Replace the title and 1st paragraph of Article A13.4.2 with the following:

A13.4.2—Decks Supporting Solid Concrete Parapet Railings and Post-and-Beam Railings with Continuous Concrete Curbs

For Design Case 1, the load *CT* applied to the deck overhang from the vehicular collision force on the railing shall be the combined force effects of transverse tensile force *T* and transverse moment M_{ct} as follows:

For portions of the overhang located further than 5 feet from a deck joint:

$$T = \frac{F_t}{10 + 2H_r + 2X_L} \qquad \text{(A13.4.2-1)}$$

$$M_{ct} = \frac{F_t H_r}{10 + 2H_r + 2X_L} \qquad \text{(A13.4.2-2)}$$

For portions of the overhang located within 5 feet of a deck joint:

$$T = \frac{F_t}{5 + H_r + X_L} \qquad \text{(A13.4.2-3)}$$

$$M_{ct} = \frac{F_t H_r}{5 + H_r + X_L} \qquad \text{(A13.4.2-4)}$$

where:

CA13.4.2

Delete the entire commentary and replace with the following:

Concrete Barrier Type 836 and Concrete Barrier Type 842 are examples of solid concrete parapet railings. Concrete Barrier Type 85 and California ST-75 Bridge Rail are examples of post-and-beam railings with continuous concrete curbs.

The effective lengths of the deck overhang used to resist the railing collision force are illustrated in Figure CA13.4.2-1. The effective lengths were determined based on the results of static nonlinear finite element analyses.

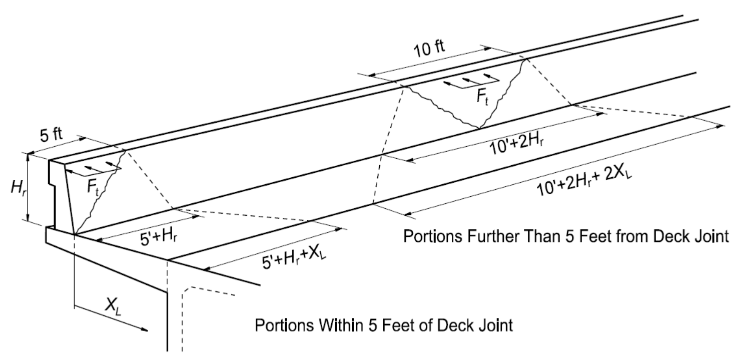

Figure CA13.4.2-1 – Effective Deck Overhang Lengths Resisting Railing Collision Force

F_t = transverse traffic railing design force from Table A13.2-1 (kips)
H_r = height of railing (ft)
M_{ct} = deck overhang transverse moment per unit length due to F_t (kip-ft/ft)
T = deck overhang transverse tensile force per unit length due to F_t (kip/ft)
X_L = transverse distance from the toe of railing to the deck overhang section being considered (ft)

The effects on the deck overhang of the longitudinal design force, F_L, from Table A13.2-1 and the effects of deck punching shear from the railing collision forces may be ignored.

The flexural resistance of the deck overhang shall be determined in accordance with Section 5, with the following additional requirements for deck overhangs on existing bridges:

- Both top and bottom transverse deck reinforcement shall be considered.
- The expected yield strength of the existing deck overhang reinforcement and the expected concrete compressive strength of the deck overhang shall be used in lieu of the specified minimum yield strength and specified concrete compressive strength, respectively.

For the expected yield strength of the existing deck overhang reinforcement and the expected concrete compressive strength of the deck overhang, see Bridge Design Memo 16.4.

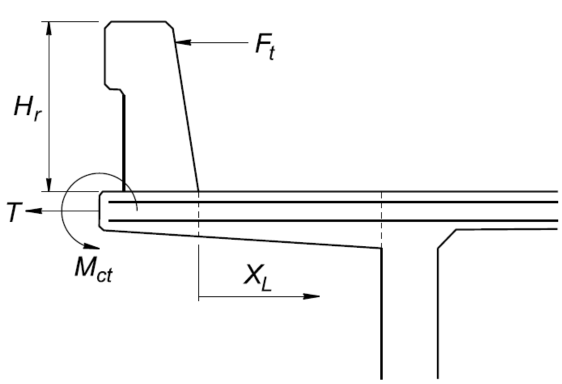

Figure A13.4.2-1 – Force Effects on Deck Overhang Due to Railing Collision Force

Replace the title of Article A13.4.3 with the following:

A13.4.3—Decks Supporting Post-and-Beam Railings Without Continuous Concrete Curbs

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14.4.1—General

Replace the 4th paragraph with the following:

For determining force effects in joints, bearings, and adjacent structural elements, the influence of their stiffnesses and expected tolerances achieved during fabrication and erection shall be considered. For design thermal movement range (Δ_T), calculations in a simply supported span, the expansion length must be taken as 0.75 of the span length.

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14.6—REQUIREMENTS FOR BEARINGS

14.6.1—General

Replace the 3rd paragraph with the following:

Bearings shall not be subjected to net uplift at any limit state.

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14.6.5.3—Design Criteria

Replace the 2nd paragraph with the following:

Bearing design shall be consistent with the intended seismic or other extreme event response of the whole bridge system. Vertical restrainers at bearing locations are not allowed.

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14.7.2.4—Contact Pressure

Replace the 3rd paragraph with the following:

Stresses shall not exceed those given in Table 14.7.2.4-1. Permissible stresses for intermediate filler contents shall be obtained by linear interpolation within Table 14.7.2.4-1. The minimum unfactored pressure on the PTFE surface must be 2.0 ksi. The maximum factored pressure on the PTFE surface must be 1.45 σ_{ss}, where σ_{ss} is the value for maximum average contact stress in Table 14.7.2.4-1.

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14.7.5—Steel-Reinforced Elastomeric Bearings—Method B

14.7.5.1—General

C14.7.5.1

Replace the 4th paragraph with the following:

The shape factor, S_i, is defined in terms of the gross plan dimensions of layer *i*. Refinements to account for the difference between gross dimensions and the dimensions of the reinforcement are not warranted because quality control on elastomer thickness has a more dominant influence on bearing behavior. Holes are not permitted in steel-reinforced bearings.

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14.7.5.3.1—Scope

Replace the 1st paragraph with the following:

Bearings designed by the provisions herein shall be tested in accordance with the requirements for steel-reinforced elastomeric bearings as specified in Article 18.2 of the AASHTO *LRFD Bridge Construction Specifications* and the AASHTO M 251. The minimum average compressive stress due to *DC* must not be less than 0.200 ksi.

Add a new paragraph after the 1st paragraph:

The maximum force at slippage must be as follows:

$$F_s = \frac{G A_r}{h_{rt}} \Delta_s \leq 0.2 P_D \qquad (14.7.5.3.1\text{-}1)$$

where:

F_s = lateral bearing force (kips)
G = shear modulus of elastomer (ksi)
A_r = reduced rubber area (in^2)
h_{rt} = total elastomer thickness (in)
P_D = compressive load at service limit state due to permanent loads (kips)
Δ_s = maximum non-seismic bearing displacement (in)

14.7.5.3.2—Shear Deformations

Replace the 1st paragraph with the following:

The maximum horizontal displacement of the bridge superstructure, Δ_O, shall be taken as 75 percent of the design thermal movement range, Δ_T computed in accordance with Article 3.12.2, combined with movements caused by creep, shrinkage, and post-tensioning.

C14.7.5.3.2

Replace the 2nd paragraph with the following:

Generally, the installation temperature is within ±25 percent of the average of the maximum and minimum design temperatures. Consequently, 75 percent of the thermal movement range is used for design purposes. The forgiving nature of elastomeric bearings more than accounts for actual installation temperatures greater than or less than the likely approximated installation temperature. Additionally, if the bearing is originally set or reset at the average of the design temperature range, 50 percent of the design thermal movement range computed in accordance with Article 3.12.2 may be substituted for 75 percent as specified.

14.7.5.3.4—Stability of Elastomeric Bearings

Replace the article with the following:

Bearings shall be investigated for instability at the strength limit load combinations specified in the Table 3.4.1-1.

The critical buckling load at strength limit displacement ($\Delta_S = \Delta_{Sst} + \Delta_{Scy}$) is given by:

$$P'_{cr_s} = P_{cr_s} \frac{A_r}{A} \qquad (14.7.5.3.4\text{-}1)$$

with

$$A_r = B(L - \Delta_s) \qquad (14.7.5.3.4\text{-}2)$$

and for rectangular bearings is:

$$P'_{cr} = 0.680 \frac{G \times B \times L^2 \times (L - \Delta_s)}{\left(1 + \frac{L}{B}\right) \times t \times T_r} \qquad (14.7.5.3.4\text{-}3)$$

A bearing design may be considered acceptable for buckling if:

$$\frac{P'_{cr_s}}{(\gamma_{DC} P_{DC} + \gamma_{DW} P_{DW}) + \gamma_L (P_{Lst} + P_{Lcy})} \geq 2.0 \qquad (14.7.5.3.4\text{-}4)$$

where:

A = bonded rubber area of elastomeric bearing (in^2)
A_r = reduced bonded rubber area of elastomeric bearing (in^2)
B = long plan dimension of rectangular bearing (in)
G = shear modulus of rubber (psi)
L = short plan dimension of rectangular bearing (in)
P_{cr_s} = critical load in un-deformed configuration (kip)
P'_{cr_s} = critical load in deformed configuration (kip)

P_{DC} = dead load (kip)
P_{DW} = wearing surfaces and utilities load (kip)
P_{Lst} = static component of live load (kip)
P_{Lcy} = cyclic component of live load (kip)
t = rubber layer thickness (in)
T_r = total rubber thickness (in)
γ_{DC} = load factor for dead load
γ_{DW} = load factor for wearing surfaces and utilities loads
γ_L = load factor is either HL93 or Permit truck load
Δ_S = non-seismic lateral displacement (in)
Δ_{Sst} = static component of non-seismic lateral displacement (in)
Δ_{Scy} = cyclic component of non-seismic lateral displacement (in)

C14.7.5.3.4

Delete the commentary.

14.8.3—Anchorage and Anchor Bolts

14.8.3.1—General

C14.8.3.1

Replace the 3rd paragraph with the following:

Uplift must be prevented both among the major elements, such as the girder, bearing, support, and between the individual components of a bearing.

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14.10—REFERENCES

Add the following reference:

Constantinou, M.C., Kalpakidis, I., Filiatrault, A and Ecker Lay, R.A. (2011), "*LRFD-Based Analysis and Design Procedures for Bridge Bearings and Seismic Isolators*," Report No. MCEER-11-0004, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

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