California Amendments to the AASHTO LRFD Bridge Design Specifications (2012 Sixth Edition)

January 2014

DEPARTMENT OF TRANSPORTATION
STATE OF CALIFORNIA

Foreward

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 1999, SCOBS voted to no longer update the *Standard Specifications for Highway Bridges*, which was the basis for the *Caltrans Bridge Design Specifications (BDS)*, and support load and resistance factor design as the primary design code. 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, Third 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, Fourth 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. 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.

CALTRANS SEISMIC DESIGN CRITERIA (CURRENT VERSION):
Shall supersede all provisions for seismic design, analysis, and detailing of bridges contained in the *AASHTO LRFD Bridge Design Specifications.* The Caltrans Seismic Design Criteria is used in conjunction with the Extreme Event I Load Combination specified in AASHTO LRFD.

The AASHTO Specifications shall be adhered to in areas where the California Specifications, design criteria, and/or the Contract Documents are silent.

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*, 6th edition with California Amendments".

LEGEND:

Amendments originating January 2014 and unchanged since that time are denoted using <u>single-underlines</u> and ~~single-strikethroughs~~.

1.3—DESIGN PHILOSOPHY

1.3.3—Ductility

Revise Article 1.3.3 as follows:

The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength ~~and extreme event~~ limit state~~s~~ 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 prevent 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 the strength limit state:~~

~~η_D ≥ 1.05 for nonductile components and connections~~

~~= 1.00 for conventional designs and details complying with these Specifications~~

~~≥ 0.95 for components and connections for which additional ductility enhancing measures have been specified beyond those required by these Specifications.~~

For all ~~othe~~r 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

Revise Article 1.3.4 as follows:

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

~~For the strength limit state:~~

~~η_R ≥ 1.05 for nonredundant members~~

~~= 1.00 for conventional levels of redundancy, foundation elements where ϕ already accounts for redundancy as specified in Section 10.5~~

~~≥ 0.95 for exceptional levels of redundancy beyond girder continuity and a torsionally-closed cross-section.~~

For all ~~other~~ limit states:

$\eta_R = 1.00$

C1.3.4

Add a new last paragraph as follows:

<u>A value of 1.0 is being used for η_R until its application is better defined.</u>

1.3.5—Operational Importance

Revise Article 1.3.5 as follows:

~~For the strength limit state:~~

η_I ~~≥ 1.05 for important bridges~~

~~= 1.00 for typical bridges~~

~~≥ 0.95 for relatively less important bridges.~~

For all ~~other~~ limit states:

η_I = 1.00

C1.3.5

Add a new last paragraph as follows:

<u>A value of 1.0 is being used for η_I until its application is better defined.</u>

This page intentionally left blank

2.3.2.2.3—Geometric Standards

Revise as follows:

Requirements of the *Caltrans Highway Design Manual* ~~AASHTO publication *A Policy on Geometric Design of Highways and Streets*~~ 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

Revise as follows:

Road surfaces on a bridge shall be given antiskid characteristics, crown, drainage, and super elevation in accordance with ~~*A Policy on Geometric Design of Highways and Streets*~~ the *Caltrans Highway Design Manual* or local requirements.

2.3.3.2—Highway Vertical

Revise the 1st and 2nd Paragraphs as follows:

The vertical clearance of highway structures shall be in conformance with the ~~AASHTO publication *A Policy on Geometric Design of Highways and Streets*~~ *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*. ~~should be 1.0 ft. greater than the highway structure clearance. The vertical clearance from the roadway to the overhead cross bracing of through truss structures should not be less than 17.5 ft.~~

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

Revise the 2nd Paragraph as follows:

Horizontal clearance under a bridge should meet the requirements of Article 2.3.2.2.1 and 2.3.2.2.3.

2.6.4.4.2—Bridge Scour

C2.6.4.4.2

Add the following after 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.

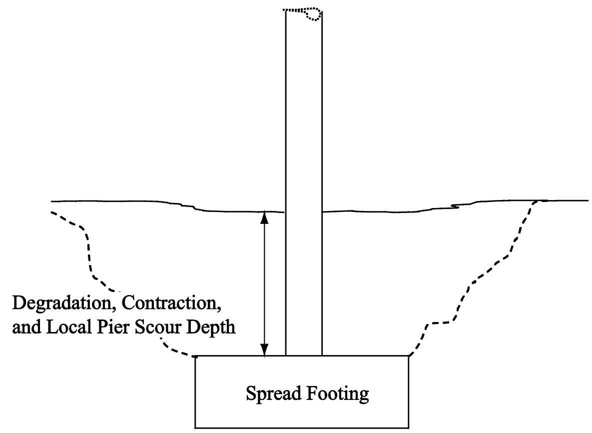


Figure C2.6.4.4.2-1—Spread Footing Location

2.6.4.4.2—Bridge Scour

Revise the 3rd Paragraph as follows:

Spread footings on soil or erodible rock shall be located so that the top of footing is below the design 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.

Revise the 4th Paragraph as follows:

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 ~~streambed~~ total scour elevation, attention shall be given to the scour potential of the design.

C2.6.4.4.2

Add a bullet to the 4th Paragraph:

- Service life for a new construction project is assumed to be 75 years.

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.

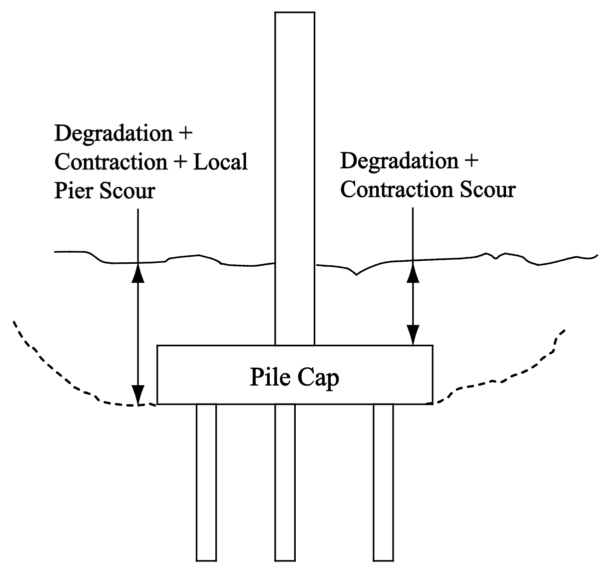


Figure C2.6.4.4.2-2—Deep Foundation Location

C2.6.6.3

Revise as follows:

For further guidance or design criteria on bridge deck drainage, see the ~~"Storm Drainage" chapter of the AASHTO *Model Drainage Manual, Policy on Geometric Design of Highways and Streets* and AASHTO/FHWA Research Report RD-87-014, *Bridge Deck Drainage Guidelines.*~~ *, Caltrans Highway Design Manual, Bridge Memo to Designers, and Bridge Design Aids.*

This is page is intentionally left blank.

3.3.2—Load and Load Definitions

Add definitions:

DC = dead load of structural components and nonstructural attachments
$DC_{Sub.}$ = dead load of structural components and nonstructural attachments of substructure
$DC_{Sup.}$ = dead load of structural components and nonstructural attachments of superstructure
DW = dead load of wearing surfaces and utilities

ES = earth surcharge load
ES_H = earth surcharge horizontal load
ES_V = earth surcharge vertical load
EV = vertical pressure from dead load of earth fill

3.3.2—Load and Load Definitions

Revise this load designation:

PS = secondary forces from post-tensioning for strength limit states; total prestress force for service limit states

3.4.1—Load Factors and Load Factor Combinations

Revise as follows:

where:

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

Revise the 2nd Bullet in the 2nd Paragraph as follows:

- STRENGTH II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

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

Revise the 2nd Paragraph C3.4.1 as follows:

~~The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.5 regarding other traffic on the bridge simultaneously~~. The vehicular braking force shall not be included in this load combination.

Revise the 4th bullet in the 2nd paragraph of Article 3.4.1:

- Strength IV – Load combination relating to very high dead load to live load force effect ratios in bridge superstructures.

Revise the 6th bullet of the 2nd paragraph of Article 3.4.1 as follows:

- 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 ordinary standard bridges $\gamma_{EQ} = 0$

Revise the last sentence of the 5th paragraph of Article C3.4.1:

This load combination is not applicable to ~~can control during~~ investigation of construction stages, substructures, earth retaining structures (including abutments), and bearing design. Other load combinations adequately address construction stages, substructures, earth retaining structures, and bearings.

Revise the 6th paragraph of C3.4.1as follows:

~~Past editions of the Standard Specifications used $\gamma_{EQ} = 0.0$. This issue is not resolved. The possibility of partial live load, i.e., $\gamma_{EQ} < 1.0$, with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that $\gamma_{EQ} = 0.50$ is reasonable for a wide range of values of average daily truck traffic (ADTT).~~ 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.

Revise the 2nd bullet of the 7th Paragraph of C3.4.1as follows:

- 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 ~~and contraction 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. ~~Alternatively, one-half of the total scour may be considered in combination with *BL*, *EQ*, *CT*, *CV*, or *IC*.~~

Revise the 3rd bullet of the 7th Paragraph of C3.4.1 as follows:

- 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.

3.4.1—Load Factors and Load Combinations

C3.4.1

Revise the 15th Paragraph as follows:

The load factor for Fatigue I load combination, applied to a single design truck, having the axle spacing specified in Article 3.6.1.4.1, reflects load levels found to be representative of the maximum stress range of the truck population for infinite fatigue life design. ~~The factor was chosen on the assumption that the maximum stress range in the random variable spectrum is twice the effective stress range caused by Fatigue II load combination~~

Add the following after the 15th Paragraph:

Infinite fatigue life is the design concept used for higher traffic volume bridges. The maximum fatigue stress range is kept lower than the constant-amplitude fatigue threshold to provide a theoretically infinite fatigue life.

A comprehensive comparison study of fatigue load moments for steel girder bridges using the AASHTO LRFD (3rd Edition, 2004) and the AASHTO Standard Specifications (17th Edition, 2002) was performed. From this parametric study, it is observed that the LRFD fatigue moments in an interior girder are about 60% and 20% less than that of the Standard for finite fatigue life and infinite fatigue life, respectively.

To reflect past Caltrans infinite fatigue life design practice using the AASHTO Standard Specifications, the load factor of 1.75 should be used in the Fatigue I Limit State. This factor is based on stress ranges due to the passage of the fatigue truck specified in Article 3.6.1.2.2 with a constant spacing of 30.0 ft between the 32.0-kip axles and derived by calibrating the LFRD fatigue design procedure to Caltrans past LFD design procedure.

Revise the 13th bullet item as follows:

- FATIGUE II -Fatigue and fracture load combination related to finite load-induced fatigue life~~.~~ due to a single P-9 design truck live load having the axle spacing specified in Article 3.6.1.4.1.

Add two bullets to the end of Paragraph 2 in Article 3.4.1:

- Construction I–Load combination related to construction condition where abutment has been built however superstructure has not been constructed.
- Construction II–Load combination related to construction condition, where soil surrounding the abutment has been removed for repair, widening or other reasons.

Revise the 16th Paragraph as follows:

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 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 ~~effective stress range 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.

This page intentionally left blank

Revise the 10th paragraph of Article 3.4.1:

The load factor for settlement, γ_{SE} , ~~should~~ shall be taken as: ~~considered on a project-specific basis In lieu of project specific information to the contrary, γ_{SE} may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement.~~

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.

Revise Table 3.4.1-1 as follows:

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	—	1.00	—	—	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
STRENGTH II	γ_p	—	1.35	1.00	—	—	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
STRENGTH III	γ_p	—	—	1.00	1.40	—	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
STRENGTH IV	γ_p	—	—	1.00	—	—	1.00	0.50/ 1.20	—	—	—	---	—	—	—
STRENGTH V	γ_p	1.35	—	1.00	0.40	1.0	1.00	0.50/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
EXTREME EVENT I	~~γ_p~~ 1.00	γEQ	—	1.00	—	—	1.00	—	—	—	1.00	---	—	—	—
EXTREME EVENT II	~~γ_p~~ 1.00	0.50	—	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
SERVICE I	1.00	1.00	—	1.00	0.30	1.0	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
SERVICE II	1.00	1.30	—	1.00	—	—	1.00	1.00/ 1.20	—	—	—	---	—	—	—
SERVICE III	1.00	0.80	—	1.00	—	—	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}	—	---	—	—	—
SERVICE IV	1.00	—	—	1.00	0.70	—	1.00	1.00/ 1.20	—	1.0	—	---	—	—	—
FATIGUE I - $LL_{HL\text{-}93}$, IM & CE ONLY	—	~~1.50~~ 1.75	—	—	—	—	—	—	—	—	—	---	—	—	—
FATIGUE II - LL_{Permit}~~,~~ IM & CE ONLY	—	~~0.75~~	1.00	—	—	—	—	—	—	—	—	---	—	—	—

Add Article 3.4.5 that includes 3.4.5.1 and 3.4.5.2 as follows:

3.4.5—Load Factors for Abutments

Abutments shall be designed for the Service, Strength and Construction limit states specified in Article 3.4.5.1.

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.

Abutments shall be designed for the Strength, and Construction load combinations, specified in Table 3.4.5.1-1.

Table 3.4.5.1-1 Strength and Construction Load Factors for Abutments

Combination	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*EH, ES_H*	*EV ES_V*	*LL_{HL93}/ BR/CE, PL, LS*	*LL_{Permit} /CE*	*WA*	*WS*	*WL*	*TU*	*PS, CR, SH*
Strength I	0.9/1.25	0.9/1.25	0.65/ 1.50	0.75/ 1.50	1.00/ 1.35	1.75	0	1.00	0	0	1.0	1.00
Strength II	0.9/1.25	0.9/1.25	0.65/ 1.50	0.75/ 1.50	1.00/ 1.35	0	1.35	1.00	0	0	1.0	1.00
Strength III	0.9/1.25	0.9/1.25	0.65/ 1.50	0.75/ 1.50	1.00/ 1.35	0	0	1.00	1.4	0	1.0	1.00
Strength V	0.9/1.25	0.9/1.25	0.65/ 1.50	0.75/ 1.50	1.00/ 1.35	1.35	0	1.00	0.4	1.0	1.0	1.00
Construction I	0	0.9/1.25	0	0.75/ 1.50	1.00/ 1.35	0	0	0	0	0	0	0
Construction II	1.25	1.25	1.50	0	0	0	0	0	0	0	1.0	1.00

3.4.5.2—Extreme Event-I (Seismic) Load Combination

If an abutment meets following height limitations seismic forces shall be considered **only** in global stability analysis of the abutment when such analysis is required:

- 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 type abutments.

Components of abutments such as shear keys are checked for seismic effects per Caltrans Seismic Design Criteria (SDC). Abutments in non-competent soil require special analysis.

Non-Integral Type Abutment (with/without piles)

Integral Type Abutment (with/without piles)

Figure 3.4.5.2-1

3.6.1.1.1—Number of Design Lanes

Revise the 1st paragraph as follows:

~~Generally,~~ Unless specified otherwise, the width of the design lanes should be taken as 12.0 ft. T~~t~~he number of design lanes should be determined by taking the integer part of the ratio w/12.0, where w is the clear roadway width in ~~ft~~ feet between curbs and/or barriers. Possible future changes in the physical or functional clear roadway width of the bridge should be considered.

Page intentionally left blank

3.6.1.3—Application of Design Vehicular Live Loads

3.6.1.3.1—General

Add a 4th bullet to the first 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 ft from the lead axle of one tandem to the rear axle of the other, combined with 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

Revise the Commentary, 3rd paragraph, as follows:

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. ~~Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. One hundred percent of the combined effect of the design tandems and the design lane load should be used.~~ 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.

This page intentionally left blank

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.

3.6.1.4.1–Magnitude and Configuration

Revise the first Paragraph as follows:

For the Fatigue I limit state, t~~T~~he 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 after the 2nd paragraph:

For the Fatigue II limit state, the fatigue load shall be one Permit truck as specified in Figure 3.6.1.4.1-2 and Figure 3.6.1.4.1-3.

C 3.6.1.4.1

Add the following Paragraph:

Fatigue Permit Truck specified in 3.6.1.4.1-2 represents the majority of permit trucks allowed in California.

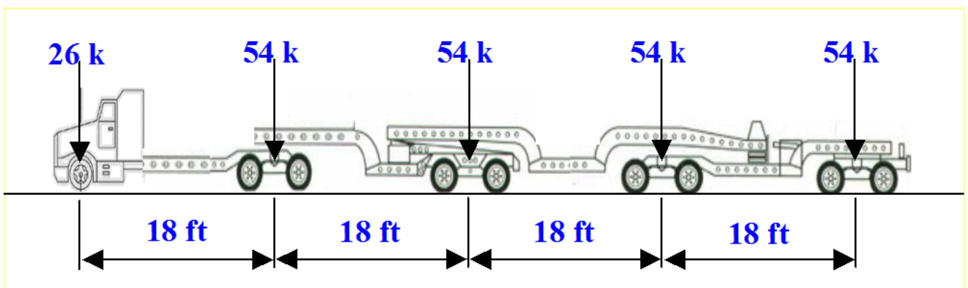


Figure 3.6.1.4.1-2 Fatigue Permit Truck

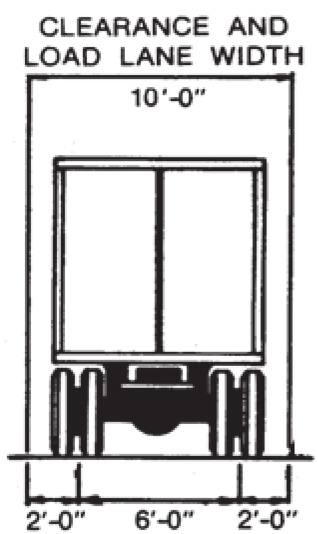


Figure 3.6.1.4.1-3 Fatigue Permit Truck

3.6.1.4.2 –Frequency

Add the following as the last paragraph:

In the absence of specific data, *ADTT* should 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 of 2500 for the HS-20 fatigue truck has been successfully used for designing new structures and widenings in California. Since stress cycles caused by an *ADTT* of 2500 are actually larger than the N_{TH} stress cycles, caused by a 75-year $(ADTT)_{SL}$ equivalent to infinite life, all bridges shall be designed for infinite load-induced 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 P9 permit truck with a volume of *ADTT* = 20.

This page intentionally left blank

3.6.1.8—Permit Vehicles

Add Article 3.6.1.8 as follows:

3.6.1.8.1 General

The weights and spacings of axles and wheels for the overload truck shall be as specified in Figures 3.6.1.8.1-1 and 3.6.1.8.1-2.

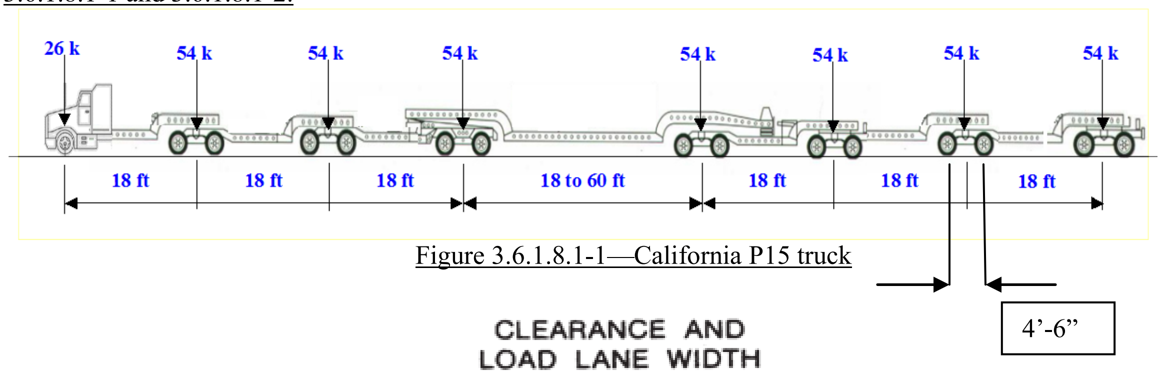


Figure 3.6.1.8.1-1—California P15 truck

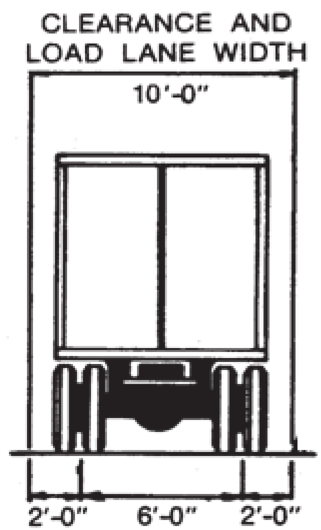


Figure 3.6.1.8.1-2 —California P15 truck gage

3.6.1.8.2—Application

The permit design live loads 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.

Dynamic load allowance shall be applied as specified in 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.

Add Commentary to Article 3.6.1.8 as follows:

C3.6.1.8

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

3.6.2—Dynamic Load Allowance: *IM*

3.6.2.1—General

Revise the 1st Paragraph as follows:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck, ~~or~~ 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.

Revise Table 3.6.2.1-1

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

C3.6.2.1

Revise paragraph 4 in Article C3.6.2.1 as follows:

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. <u>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 P-loads on short-span bridges.</u>

Revise the 6th Paragraph as follows:

~~For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted.~~ 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.

3.6.3—Centrifugal Forces: *CE*

Revise Paragraph 1, as follows:

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

(no change to equation)

Revise Paragraph 2, as follows:

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

Revise Paragraph 4, as follows:

Centrifugal forces ~~shall~~ may be applied horizontally at a distance 6.0 ft above the roadway surface.

In the Commentary, C3.6.3, revise Paragraph 4, as follows:

Centrifugal force ~~also~~ causes an overturning effect on the wheel loads when ~~because~~ 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*

Revise Paragraph 1, Sentence 3, as follows:

….These forces shall be assumed to act horizontally at ~~a distance of 6.0 ft above~~ the roadway surface in either longitudinal direction to cause extreme force effects….

In C3.6.4, add a sentence to the end of paragraph one, as follows:

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.

3.6.5—Vehicular Collision Force: *CT*

3.6.5.1—Protection of Structures

Modify the first paragraph as follows:

Unless the Owner determines that site conditions indicate otherwise, ~~abutments~~ bents and piers located within a distance of 30.0 ft to the edge of roadway shall be investigated for collision. Collision shall be addressed by either providing structural resistance or by redirecting or absorbing the collision load. The provisions of article 2.3.2.2.1 shall apply as appropriate.

Modify the second paragraph as follows:

Where the design choice is to provide structural resistance, the pier ~~or abutment~~ shall be designed for an equivalent static force of 600 kip, which is assumed to act in any direction ~~of zero to 15 degrees with the edge of the pavement~~ in a horizontal plane, at a distance of 5.0 ft above ground.

Add the following paragraphs after the 2nd paragraph:

Where the design choice is to provide structural resistance, the goal is to prevent collapse. The resistance of the loaded component shall be based on strain using expected material properties and equilibrium and strain compatibility as defined in the Caltrans Seismic Design Criteria. The axial compression in the column/pier for this evaluation shall be based on dead load (*DC*) only with a load factor of 1.0.

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

This page intentionally left blank

3.7.5—Change in Foundation Due to Limit state for Scour

Revise Article 3.7.5 as follows:

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.~~at strength and service limit states. The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit states.~~

Revise the 2nd paragraph of the Commentary as follows:

Provisions concerning the effects of scour are given in Section 2. Scour per se is not a force effect, 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.

This page intentionally left blank

3.8.1.3—Wind Pressure on Vehicles: *WL*

Revise Article 3.8.1.3, Paragraph 1, as follows:

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles may ~~shall~~ be represented by ~~an interruptible, moving~~ a continuous force of 0.10 klf acting normal to, and 6.0 ft above the roadway and shall be transmitted to the structure.

Add a new 3rd paragraph to the Commentary, C3.8.1.3, 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. If the load is applied at deck level rather than 6.0 ft above the deck, the effect on member sizes and quantities is generally negligible.

This page intentionally left blank

3.10 EARTHQUAKE EFFECTS: *EQ*

Delete Article 3.10 in its entirety and replace with the following:

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.

This page intentionally left blank

Revise Article 3.12 as follows:

3.12—FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: *TU, TG, SH, CR, SE, PS*

3.12.2 —Uniform Temperature

The design thermal movement associated with a uniform temperature change ~~may~~ shall be calculated using Procedure A. ~~or Procedure B below. Either Procedure A or Procedure B may be employed for concrete deck bridges having concrete or steel girders. Procedure A shall be employed for all other bridge types~~.

3.12.2.1—Temperature Range for Procedure A

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

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.

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

$$\Delta_T = +/- \alpha L(T_{maxDesign} - T_{minDesign})/2 \quad \text{(Eq. 3.12.2.1-1)}$$

where:

L = expansion length (in.)

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

C3.12.2.1

Add paragraph 4 as follows:

Expansion length is defined as the distance from the point of no thermal movement to the point under consideration (usually a joint or bent location).

3.12.2.2—Temperature Range for Procedure B

Delete contents of the entire Article including Commentary and Figures.

3.12.2.3—Design Thermal Movements

Revise as follows:

The design thermal movement range, Δ_T, for joints and bearings, shall be used in conjunction with the higher value for γ_{TU} and depend upon the extreme bridge design temperatures defined in Article 3.12.2.1 ~~or 3.12.2.2~~ and be determined as:

$$\Delta_T = \alpha L(T_{maxDesign} - T_{MinDesign}) \qquad \text{(Eq. 3.12.2.3-1)}$$

where:

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

Add 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 joints and bearings 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.

This page intentionally left blank

4.3—NOTATION:

Add the following definitions:

I_{cr} = moment of inertia of the cracked 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)

This page intentionally left blank

4.4—ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS:

Delete the 3rd Paragraph as follows:

~~The name, version, and release date of software used should be identified in the contract documents.~~

This page intentionally left blank

C4.5.2.2:

Add a 2nd Paragraph as follows:

Analytical studies have been performed to determine the effects of using gross and cracked moment of inertia sectional properties (I_{gs} & I_{cr}) of concrete columns. The Caltrans studies yielded the following findings on prestressed concrete girders on concrete columns:

1. Using I_{gs} or I_{cr} in the columns has minor effects on the superstructure moment and shear demands from external vertical loads. Using I_{gs} or I_{cr} in the columns will significantly affect the superstructure moment and shear demands from thermal and other lateral loads.
2. Using I_{cr} in the columns can reduce column force and moment demands.
3. Using I_{cr} in the columns can increase the superstructure deflection and camber.

C4.5.2.3:

Add a 4th Paragraph as follows:

For reinforced concrete columns supporting non-segmental bridge structures, engineers may use an estimated cracked moment of inertia for the respective column sections. The cracked properties may be incorporated into the structural models to analyze non-seismic force demands. Engineers may use methods prescribed in Section 5 for the estimated cracked moment of inertia.

This page intentionally left blank

4.6.1.1—Plan Aspect Ratio

Revise the 2nd Paragraph as follows:

The length-to-width restriction specified above does not apply to ~~cast-in-place multi-cell box girders~~ concrete box girder bridges.

This page intentionally left blank

4.6.2.2.1—Application

Revise the 1st and 6th Paragraphs as follows:

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.

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

4.6.2.2.1 —Application

Revise the 9th Paragraph as follows:

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 coincident shears.

C4.6.2.1.1

Revise the 8th Paragraph, as follows.

Whole-width design is appropriate for torsionally-stiff cross-sections where load-sharing between girders is extremely high and torsional loads are hard to estimate. Prestressing force should be evenly distributed between girders. Cell width-to-height ratios should be approximately 2:1. The distribution factors for exterior girder moment and the two or-more-lanes loaded distribution factors for exterior girder shear are not used because using the distribution factors for interior girders would provide a conservative design. In general, the total number of design lanes doesn't change appreciably when using interior girders distribution factors for the whole-widths. The one-design-lane-loaded distribution factor for exterior girder shear is not used because lever rule isn't appropriate for use in multi-cell boxes.

Add the following:

C4.6.2.2.2

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

Revise the following:

4.6.2.2.2.b-i Interior Beams with Concrete Decks

Add the following:

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

For cast-in-place concrete box girder shown as cross-section type "d", the live load distribution for moment on one-cell and two-cell (Nc = 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.b-2 where the moment reinforcement shall be distributed equally across the total bridge width (within the effective flanges).

Add the following:

C4.6.2.2.2b-ii

The Caltrans Structural Analysis Committee conducted parametric studies on one-cell and two-cell box girder bridges using SAP2000 3D 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

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

This page intentionally left blank

Add the following 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*	
		$\frac{W}{12}(1.65 - 0.01W)$**	$6 \le W < 10$
		1.3	$10 \le W \le 24$
		Any Fraction or Number of Lanes:	
		$\frac{W}{12}(1.65 - 0.01W)$**	$6 \le W < 12$
		$\frac{W}{12}(1.5 - 0.014W)$	$12 \le W < 20$
		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: $\frac{W}{12}(1.5 - 0.014W)$	$12 \le W \le 36$

* 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.

This page intentionally left blank

4.6.2.2.2e — Skewed Bridges

Revise the 1st Paragraph as follows:

~~When the line supports are skewed and the difference between skew angles of two adjacent lines of supports does not exceed 10 degrees, the bending moment in the beams may be reduced in accordance with Table 4.6.2.2.e-1.~~ Caltrans presently does not take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.

C4.6.2.2.2e

Revise the 1st Paragraph as follows

~~Accepted reduction factors are not currently available for cases not covered in Table 4.6.2.2.e-1.~~

This page intentionally left blank

Add the following:

C4.6.2.2.3

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

Revise the following:

4.6.2.2.3.a-i Interior Beams

Add the following:

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

For cast-in-place concrete box girder shown as cross-section type "d", the live load distribution for shear on one-cell and two-cell (Nc = 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 the shear reinforcement shall be equally distributed to each girder web (for non-skew conditions).

Add the following:

C4.6.2.2.3a-ii

The Caltrans Structural Analysis Committee conducted parametric studies on one-cell and two-cell box girder bridges using SAP2000 3D 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

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

This page intentionally left blank

Add the following after Table 4.6.2.2.3a-1:

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

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 $2 \cdot \left(\frac{S}{4}\right)^{0.4} \left(\frac{d}{12L}\right)^{0.06}$	$60 \leq L \leq 240$ $35 \leq d \leq 110$ $N_c = 1$ $6 \leq S \leq 14$
		Two-Cell Box Girder $3 \cdot \left(\frac{S}{4.8}\right)^{0.5} \left(\frac{d}{12L}\right)^{0.09}$	$60 < L < 240$ $35 < d < 110$ $N_c = 2$ $6 \leq S \leq 14$

This page intentionally left blank

4.6.2.2.3c —Skewed Bridges

Revise as follows:

Shear in the exterior and first interior beams on ~~at~~ the obtuse side ~~corner~~ of the bridge shall be adjusted when the line of support is skewed. The ~~value of the~~ correction factor values for exterior and first interior beams shall be obtained from Table 4.6.2.2.3c-1. It is applied to the lane fraction specified in Table 4.6.2.2.3a-1 for interior beams and in Table 4.6.2.2.3b-1 for exterior beams. The shear correction factors are applied to girders of interests between the point of support and midspan. This factor should not be applied in addition to modeling skewed supports.

~~In determining the end shear in multibeam bridges, the skew correction at the obtuse corner shall be applied to all the beams.~~

C 4.6.2.2.3c

Add the following:

The factors in Table 4.6.2.2.3c-1 may decrease linearly to a value of 1.0 at midspan, regardless of end condition.

Revise Table 4.6.2.2.3c-1 as follows:

Table 4.6.2.2.3c-1—Correction Factors for Load Distribution Factors for Support 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, 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	$1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$ For exterior girder $1.0 + \left(0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta\right)/6$ For first interior girder of T-Sections	$0^{o} < \theta \leq 60^{o}$ $3.5 < S \leq 16.0$ $20 \leq L \leq 240$ $N_b \geq 4$
Cast-in-place Concrete Multicell Box	d	$\cancel{1.0 + \left(0.25 + \frac{12.0L}{70d}\right)\tan\theta}$ $1.0 + \frac{\theta}{50}$ for exterior girder $1.0 + \frac{\theta}{300}$ for first interior girder	$0^{o} < \theta \leq 60^{o}$ $6.0 < S \leq 13.0$ $20 \leq L \leq 240$ $35 \leq d \leq 110$ $N_c \geq 3$
Concrete Deck on Spread Concrete Box Beams	b,c	$1.0 + \frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan\theta$	$0^{o} < \theta \leq 60^{o}$ $6.0 < S \leq 11.5$ $20 \leq L \leq 140$ $18 \leq d \leq 65$ $N_b \geq 3$
Concrete Box Beams Used in Multibeam Decks	f,g	$1.0 + \frac{12.0L}{90d}\sqrt{\tan\theta}$	$0^{o} < \theta \leq 60^{o}$ $20 \leq L \leq 120$ $17 \leq d \leq 60$ $35 \leq b \leq 60$ $5 \leq N_b \leq 20$

4.6.2.2.5 - Special Loads with Other Traffic

Revise the 1st Paragraph as follows:

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 either:

- 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 4.6.2.2.2.d has been utilized for simplified analysis.
- <u>Two identical permit vehicles in separate lanes are used, as specified in CA amendment to Article 3.4.1.</u>

Add the following:

4.6.2.2.6 Permanent Loads Distribution

4.6.2.2.6a- Structural Element Self-Weight

Except for 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) and 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.

4.6.2.2.6b- Non-Structural Element Loads

Non-structural loads apply to appurtenances, utilities, wearing surface, future overlays, earth cover, and planned widenings. 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. Barriers with soundwalls that constitute significant loads, e.g., concrete or masonry walls, 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 similar to live load shears in Article 4.6.2.2.3c.

4.6.2.5 - Effective Length Factor, *K*

Revise as follows:

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

In the absence of more refined analysis, where lateral stability is provided by diagonal bracing or other suitable means, the effective length factor in the braced plane, *K*, for compression members shall be taken as unity, unless structural analysis shows a smaller value may be used. In the absence of a more refined analysis, the effective length factor in the braced plane for steel in ~~triangulated trusses,~~ trusses ~~and frames~~ may be taken as:

- For compression chords: $K = 1.0$
- For bolted or welded end conditions at both ends: $K =$ 0.85 ~~0.75~~
- ~~For pinned connections at both ends: K = 0.875~~
- For single angles regardless of end connections: $K = 1.0$

Vierendeel trusses shall be treated as unbraced frames.

C 4.6.2.5

Revise the 1st and 2nd Paragraphs as follows:

Equations for axial ~~the compressive~~ resistance of columns and moment magnification factors for beam-columns include a factor, *K*, which is used to modify the length according to the restraint at the ends of the column against rotation and translation.

K is a factor that when multiplied by the actual length of the end-restrained compression member, gives the length of an equivalent pin-ended compression member whose buckling load is the same as that of the end-restrained member. The Structural Stability Research Council (SSRC) Guide (Galambos 1988) recommends $K = 1.0$ for compression chords on the basis that no restraint would be supplied at the joints if all chord members reach maximum stress under the same loading conditions. It also recommends $K = 0.85$ for web members of trusses supporting moving loads. The position of live load that produces maximum stress in the member being designed also results in less than maximum stress in members framing into it, so that rotational restraint is developed. ~~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 the Structural Stability Research Council, are given in table C4.6.2.5-1 for some idealized column end conditions.

This page intentionally left blank

4.6.2.6—Effective Flange Width

4.6.2.6.1— General

Revise the 3rd Paragraph as the follows:

The slab effective flange width in composite girder and/or stringer system or in the chords of composite deck trusses may be taken as: ~~one- half the distance to the adjacent stringer or girder on each side of the component, or one-half the distance to the adjacent stringer or girder plus the full overhang width.~~

If $S/L \leq 0.32$, then:

$$b_e = b \qquad (4.6.2.6.1\text{-}2)$$

Otherwise:

$$b_e = \left[1.24 - 0.74\left(\frac{S}{L}\right)\right] b \geq b_{\min} \qquad (4.6.2.6.1\text{-}3)$$

where

b = full flange width (ft)
b_e = effective flange width (ft)
b_{min} = minimum effective flange width (ft)
L = span length (ft)
S = girder spacing (ft)

For interior girders, the minimum effective flange width, b_{min} may be taken as the least of:

- One-quarter of the effective span length;
- 12.0 times the average deck slab depth, plus the greater of web thickness or one-half the girder top flange width.

For exterior girders, the minimum effective flange width, b_{min} may be taken as one-half the effective width of the adjacent interior girder, plus the least of:

- One-eighth of the effective span length;
- 6.0 times the average deck slab depth, plus the greater of one-half the web thickness or one-quarter of the girder top flange width.

Otherwise, the slab effective flange width should be determined by a refined analysis when:

C4.6.2.6.1

Insert the following paragraphs after the 2nd Paragraph.

Eqs. (4.6.2.6.1-2) and (4.6.2.6.1-3) are based on state-of-the-art research by Chen, et al. (*2005*), Nassif et al. (*2005*), and Caltrans revisions. The concrete deck slabs shall be designed in accordance with Article 9.7.

The girder spacing and the full flange width are shown in Figure C4.6.2.6.1-1. For interior girders, the girder spacing, *S*, and the full flange width, *b*, shall be taken as the average spacing of adjacent girders. For exterior girders, the girder spacing, *S*, and the full flange width, *b*, shall be taken as the overhang width plus one-half of the adjacent interior girder spacing, and shall be limited to the adjacent interior girder spacing.

$$b = S = \frac{S_1}{2} + \frac{S_2}{2} \qquad b = S = \frac{S_1}{2} + S_o \leq S_1$$

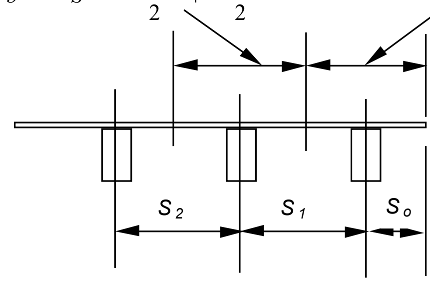


Figure C4.6.2.6.1-1 Girder Spacing and Full Flange Width.

The full flange width is proposed within the limits of the parametric study ($S \leq 16$ ft, $L \leq 200$ ft, $\theta \leq 60^o$) by Chen et al. (*2005*) based on an extensive and systematic investigation of bridge finite element models. The full flange width is also proposed within the limit of $S/L \leq 0.25$ by Nassif et al. (*2005*). For $S/L > 0.25$, Nassif et al. (*2005*) recommends that:

$$\frac{b_e}{b} = 1.0 - 0.5\left(\frac{S}{L}\right) \qquad \text{(C4.6.2.6.1-1)}$$

Figure C4.6.2.6.1-2 shows a graphic illustration of Eqs 4.6.2.6.1-2 and 4.6.2.6.1-3 which are a good combination of the effective flange width criteria proposed by Chen et al. (*2005*) and Nassif et al. (*2005*). For $S/L \leq 0.32$, the exact parametric study limit adopted by Chen et al. (*2005*), Eq. 4.6.2.6.1-2 gives the full flange width. For $S/L = 1$, Eq. 4.6.2.6.1-3 provides one-half of the full flange width which is as same as Equation C4.6.2.6.1-1.

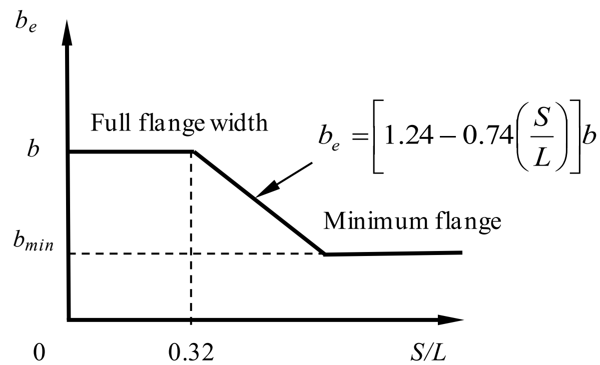


Figure C4.6.2.6.1-2 Effective Flange Width.

When $S/L > 0.32$, the effective flange width calculated by Eq. 4.6.2.6.1-3 is less than the full flange width as shown in Figure C4.6.2.6.1-2. When $S/L >$ 1.68, the effective flange width calculated by Eq. 4.6.2.6.1-3 is less than zero. A meaningful minimum effective flange width, b_{min}, based on past successful practice, is added in Eq. 4.6.2.6.1-3. The minimum effective flange width, b_{min} should be checked when S/L $>$ 0.32.

4.6.3.1-—General

Revise the 2nd Paragraph as follows:

~~A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be consider to be structurally active at service and fatigue limit states.~~ 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

Revise the 2nd Paragraph as follows:

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.

This page intentionally left blank

4.6.3.2.1- General

Revise the 1st Paragraph as follows:

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.

This page intentionally left blank

4.9—REFERENCES:

Add the following reference:

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.

Revise the following reference:

Nassif, H., A.-A. Talat, and S. El – Tawil. 200~~6~~5. “Effective Flange Width Criteria for Composite Steel Girder Bridges.” Annual Meeting CD-ROM, Transportation Research Board, National Research Council, Washington, D.C.

This page intentionally left blank

5.3 — NOTATION

Revise the definition of A_ℓ:

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

This page is intentionally left blank

5.3 — NOTATION

Revise the following definition:

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

This page is intentionally left blank.

5.4.2.1 — Compressive Strength

Revise the 3rd paragraph as follows:

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

This page is intentionally left blank.

5.4.6.2 — Size of Ducts

Modify the 2nd Paragraph as follows:

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

5.5.3.1 — General

Revise the 2nd Paragraph of Article as follows:

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

C5.5.3.1

Revise the 3rd Paragraph of Article as follows:

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

5.5.3.1 — General

Revise the 5th paragraph of Article as follows:

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

5.5.3.2 — Reinforcing Bars

Revise the 2nd Paragraph of Article as follows:

where:

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

5.5.3.4 — Welded or Mechanical Splices of Reinforcement

Revise the 1st Paragraph of Article as follows:

For welded or mechanical connections that are subject to repetitive loads, resulting from the Fatigue I load combination for infinite fatigue life, and the Fatigue II load combination for finite fatigue life specified in Table 3.4.1-1, the constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, shall be as given in Table 5.5.3.4-1.

This page is intentionally left blank.

5.5.4.2.1 Conventional Construction

Insert the following under the first bullet:

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

Modify the second bullet:

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

This page is intentionally left blank.

C5.5.4.2.1

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

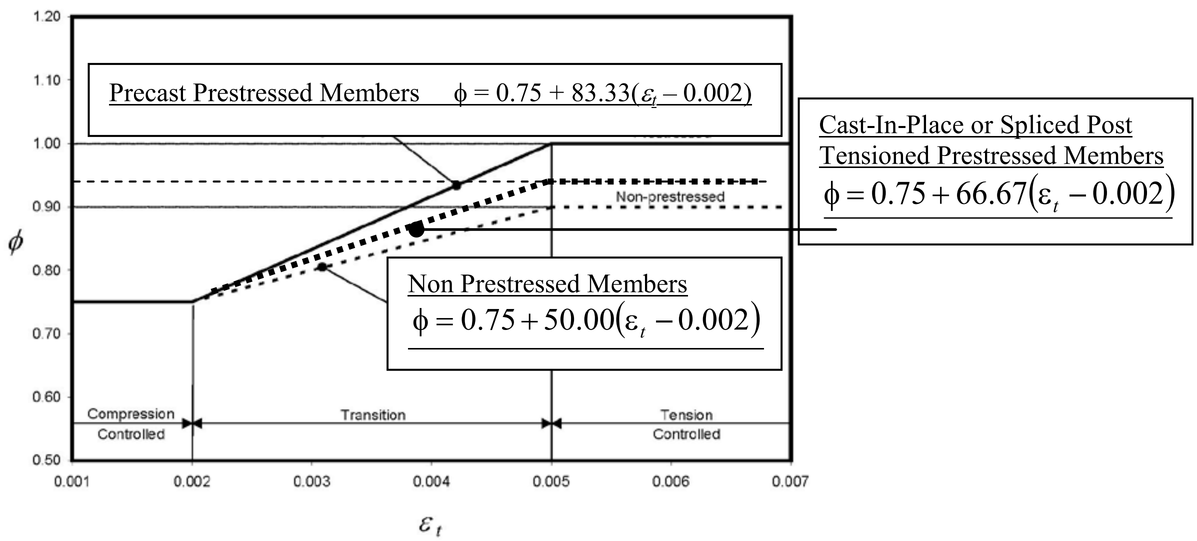


Figure C5.5.4.2.1-1 – Variation of ϕ with Net Tensile Strain ε_t ~~and d_t/c~~ for Grade 60 Reinforcement and for Prestressed~~ing~~ Members.

This page is intentionally left blank.

5.5.5 — Extreme Event Limit State

Revise as follows:

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

5.6.3.1 — General

Revise the 2nd Paragraph as follows:

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

This page is intentionally left blank.

5.7.2.1 — General

Revise the 11th "bullet" as follows:

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. <u>For non-prestressed concrete members with factored axial compressive load less than 0.10 $f'_c A_g$, the net tensile strain in the extreme tension steel at a section shall not be less than 0.004 just as the concrete in compression reaches its assumed strain limit of 0.003.</u>

C5.7.2.1

Revise the 4th Paragraph as follows:

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. <u>Sections with a factored axial compressive load that is less than 0.1$f'_c A_g$ can be regarded as flexural members</u>. Ensuring that the net tensile strain in the extreme tensile steel is not less than 0.004 is equivalent to the previously established practice of limiting the maximum reinforcement ratio in a cross section to 0.75 times the balanced reinforcement ratio. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

This page is intentionally left blank.

5.7.3.3.2 — Minimum Reinforcement

Revise the following definition:

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

This page is intentionally left blank.

5.7.3.4 — Control of Cracking by Distribution of Reinforcement

Revise the 3rd Paragraph as follows:

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

Add a new paragraph after the 3rd Paragraph:

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

This page is intentionally left blank.

5.7.3.6.2 — Deflection and Camber

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

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

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

C5.7.3.6.2

Revise the 1st Paragraph as follows:

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

Add a new 2nd Paragraph as follows:

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

5.7.3.6.2 — Deflection and Camber

Delete the 5th Paragraph and replace with the following:

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

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

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

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

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

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

where:

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

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

C5.7.3.6.2

Revise the last Paragraph as follows:

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

5.8.2.1 — General

Revise the 4th Paragraph as follows:

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

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

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

For solid sections:

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

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

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

And

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

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

where:

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

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

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

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

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

C5.8.2.1

Revise the 7th Paragraph as follows:

In box girders, torsion introduces shear forces in the webs as well as in the top and bottom slab. In most box girder sections, the torsional shear in interior girder webs will be negligible and is primarily resisted by exterior girders. ~~For a box girder, t~~The shear flow due to torsion is added to the shear flow due to flexure in one exterior web, and subtracted from the opposite exterior web. In the controlling web, the second term in Eq. 5.8.2.1-7 comes from integrating the distance from the centroid of the section, to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying by the web height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal to that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal.

Add 8th Paragraph as follows:

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

Add 9th Paragraph as follows:

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

5.8.2.4 — Regions Requiring Transverse Reinforcement

Add the following to the end of the Article 5.8.2.4:

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

This page is intentionally left blank.

5.8.2.7 — Maximum Spacing of Transverse Reinforcement

Revise the 1st bullet as follows:

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

C5.8.2.7

Add a 2nd Paragraph as follows:

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

This page is intentionally left blank.

5.8.2.9 — Shear Stress on Concrete

Revise the 2nd Paragraph as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts up to a maximum of 2 in. or one-quarter the diameter of grouted ducts up to a maximum of 1 in. at that level shall be subtracted from the web width for spliced precast girders. It is not necessary to reduce b_v for the presence of ducts in fully grouted cast-in-place box girder frames.

C5.8.2.9

Revise the 1st Paragraph as follows:

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

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

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

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

This page is intentionally left blank.

5.8.3.4 — Procedures for Determining Shear Resistance

Revise the 1st Paragraph as follows:

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

C5.8.3.4

Revise the 1st Paragraph as follows:

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

This page is intentionally left blank.

5.8.3.4.2 — General Procedure

Delete the entire Article and revise as follows:

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

C5.8.3.4.2

Delete the entire Article and revise as follows:

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

This page is intentionally left blank.

5.8.3.4.3 — Simplified Procedure for Prestressed and Nonprestressed Sections

Delete entire Article 5.8.3.4.3 and replace with the following:

Article 5.8.3.4.3 "Simplified Procedure for Prestressed and Nonprestressed Sections" shall not be used.

This page is intentionally left blank.

C5.8.3.5

Add a new 1st Paragraph as follows:

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

This page is intentionally left blank.

Add a new article:

C5.8.3.6.2

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

C5.8.3.6.3

Add the following as the 2nd Paragraph:

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

This page is intentionally left blank.

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

Revise the Last Paragraph as follows:

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

This page is intentionally left blank.

5.8.6 — Shear and Torsion for Segmental Bridges

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

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

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

This page is intentionally left blank.

Table 5.9.3-1 Stress Limitations for Prestressing Tendons

Revise Table 5.9.3-1 as follows:

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

This page is intentionally left blank.

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

Revise Table 5.9.4.2.2-1 as follows:

Bridge Type	Location	Stress Limit
Segmental and Non-Segmental Bridges	Precompresssed 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	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections • For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and/or are located in Caltrans' Environmental Areas I or II. • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and/or are located in Caltrans' Environmental Area III. • For components with unbonded prestressing tendons.	$0.19\sqrt{f'_c}$ (ksi) $0.0948\sqrt{f'_c}$ (ksi) No tension
Segmentally Constructed Bridges	(*no changes*)	(*no changes*)

This page is intentionally left blank.

5.9.5.2.2b — Post-tensioned Construction

Revise Table 5.9.5.2.2b-1 as follows:

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

C5.9.5.2.2b

Add a new last Paragraph as follows:

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

This page is intentionally left blank.

5.9.5.2.3b — Post-Tensioned Members

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

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

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

C5.9.5.2.3b

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

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

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

This page is intentionally left blank.

5.9.5.3 — Approximate Estimate of Time-Dependent Losses

Add a new last paragraph:

For cast-in-place post-tensioned members, the approximate estimate of time-dependent losses may be taken as a lump sum value of 20 ksi.

C5.9.5.3

Add a new last paragraph:

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretensioned members and should not be used for post-tensioned structures. Research performed by the University of CA, San Diego (SSRP-11/02) indicates time-dependent losses for cast-in-place post-tensioned box girder bridges are lower than previously expected. A parametric study by Caltrans using equations presented in the aforementioned research indicates losses may range from 11 ksi to 21 ksi. The variance is due to several parameters, such as relative humidity, area of non-prestressing steel and strength of concrete.

This page is intentionally left blank.

C5.10.4.3.1b

Revise the Paragraph as follows:

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

Replace Figure C5.10.4.3.1b-1 as follows:

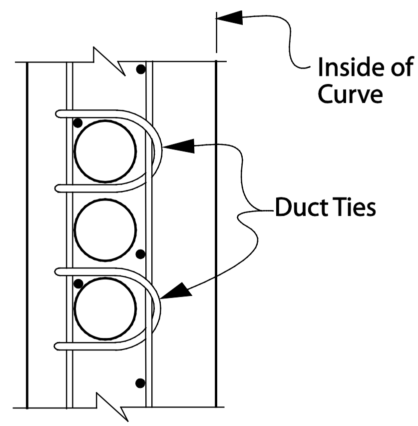


Figure C5.10.4.3.1b-1-Typical ~~Stirrup and~~ Duct Tie Detail

This page is intentionally left blank.

5.10.5 — External Tendon Supports

Add the following to the end of the Article 5.10.5:

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

C5.10.5

Add the following:

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

This page is intentionally left blank.

5.11.4.3 — Partially Debonded Strands

Revise the 2nd and 3rd Paragraphs as follows:

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

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

This page is intentionally left blank

5.12.3 — Concrete Cover

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

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

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

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

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

The concrete cover in structural elements that are in direct contact with ocean spray shall be based on the requirements for a chloride concentration greater than 10,000 ppm in the corrosive splash zone.

C5.12.3

Delete the existing text, and replace with the following:

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

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

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

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

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

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

Delete Table 5.12.3-1 and replace with the following:

Table 5.12.3-1 Minimum Concrete Cover to Reinforcement (inches) for 75 - year Design Life

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

General Notes:

1. Supplementary cementitious materials (SCM) are required for all exposure conditions period.
2. For protection of bundled bars, ducts and /or prestressing steel, see Articles 5.12.3-1, 5.12.3-2 and 5.12.3-3.
3. The minimum cover at the corners, beveled edges, and curved surfaces shall be the same as that in the corresponding members.
4. For rebar cover in CIDH piles, also refer to Table 10.8.1.3-1.

Footnotes:

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

Add this new article:

5.12.3.1 — Protection for Bundled Bars

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

Add this new article:

5.12.3.2 — Protection for Prestressing Tendons

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

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

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

C5.12.3.2

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

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

Add this new article:

5.12.3.3 — Protection for Ducts

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

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

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

This page is intentionally left blank.

5.12.4 — Protective Coatings

Delete and replace with the following:

5.12.4 Protection of Exposed Metals

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

5.12.5 — Protection for Prestressing Tendons

Delete and replace with the following:

5.12.5 Protection of Concrete Exposed to Acids and Sulfate

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

This page is intentionally left blank.

5.13.4.5.2 — Reinforcing Steel

Revise the 3rd paragraph of Article 5.13.4.5.2 as follows:

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

Add following paragraph to the end of Article 5.13.4.5.2:

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

This page is intentionally left blank.

5.14.1.4.1 — General

Modify the 1st Paragraph as follows:

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

C5.14.1.4.1

Add a new 1st Paragraph as follows:

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

This page is intentionally left blank.

B5.1 — BACKGROUND

Revise the 1st Paragraph as follows:

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

B5.2— Sectional Design Model General Procedure

Add the following after the 5th Paragraph:

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

CB5.2 Sectional Design Model General Procedure

Add the following after the 8th Paragraph:

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

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

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

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

6.4.3.1—Bolts

Revise the 3rd Paragraph as follows:

AASHTO M 253 (AASHTO A490) bolts and ASTM A354 Grade BD, bolts, studs, and other externally threaded fasteners, ASTM F1554 Grade 105 (with F_u = 150 ksi) anchor bolts and ASTM A722 bars shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C6.4.3.1

Add a 2nd Paragraph as follows:

Galvanization of AASHTO M 253 (ASTM A490) and ASTM A354 Grade BD fasteners, ASTM F1554 Grade 105 (with F_u = 150 ksi) anchor bolts and ASTM A722 bars is not permitted due to hydrogen embrittlement. These fasteners should be carefully used with applicable protective coatings conforming to AASHTO M 253 (ASTM A490) and ASTM A354, ASTM F1554 and ASTM A722 Specifications.

This is page is intentionally left blank.

6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1.2.3— Detail Categories

Revise Table 6.6.1.2.3-2 as follows:

Table 6.6.1.2.3-2—N_{TH} and 75-yr $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	N_{TH} (Number of Cycles Equivalent to Infinite Life)	75-yrs $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per Day)
A	1,809,000	~~530~~ 65
B	2,930,000	~~860~~ 110
B′	3,530,000	~~1035~~ 130
C	4,400,000	~~1290~~ 160
C′	2,546,000	~~745~~ 90
D	6,413,000	~~1875~~ 230
E	12,071,000	~~3530~~ 440
E′	22,189,000	~~6485~~ 815

C6.6.1.2.3

Revise the first sentence of the 8th Paragraph as follows:

The values in the second and the third columns of Table 6.6.1.2.3-2 were computed as follows:

$$\cancel{75\_Year(ADTT)_{SL} = \frac{A}{\left[\frac{(\Delta F)_{TH}}{2}\right]^3 (365)(75)(n)}}$$

$$\underline{75\_Year(ADTT)_{SL} = \frac{N_{TH}}{(365)(75)(n)}} \qquad \text{(C6.6.1.2.3-1)}$$

$$\underline{N_{TH} = \frac{A}{\left[(\Delta F)_{TH}\right]^3}} \qquad \underline{\text{(C6.6.1.2.3-2)}}$$

This page is intentionally left blank.

6.6.1.2.5 —Fatigue Resistance

Revise Table 6.6.1.2.5-2 as follows:

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

Longitudinal Members		Span Length	
		> 40.0 ft.	≤ 40.0 ft.
Simple Span Girders		1.0	2.0
Continuous Girders	Near Interior Support	1.5 (Fatigue I) 1.2 (Fatigue II)	2.0
	Elsewhere	1.0	2.0
Cantilever Girders		5.0	
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling		5.0	
Trusses		1.0	
Transverse Members		Spacing	
		> 20.0 ft.	≤ 20.0 ft.
		1.0	2.0

C6.6.1.2.5

Add a new last Paragraph as follows:

Cycles per design fatigue Permit Truck (Fatigue II limit state) passage are evaluated by the rainflow method. The numbers of cycles induced by the fatigue Permit Truck passage are somewhat similar to the cycles induced by the HL-93 fatigue truck used for Fatigue I Limit State, except in the case of near interior supports of bridges that spans greater than 40 feet.

This page is intentionally left blank.

6.10.7.1.2—Nominal Flexural Resistance

Revise Eq. 6.10.7.1.2-2 as follows:

$$\cancel{M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)}$$

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{\frac{D_p}{D_t} - 0.1}{0.32}\right)\right]M_p \qquad (6.10.7.1.2\text{-}2)$$

C6.10.7.1.2

Revise the 2nd Paragraph as follows:

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 only on the plastic moment resistance M_p, the yield moment resistance M_y, and ~~on~~ the ratio D_p/D_t. ~~as also suggested in Yakel and Azizinamini (2005). Both equations implement the above philosophy justified by Wittry (1993). Eq. 10.7.1.2-2 is somewhat more restrictive than the equation in previous Specifications for sections with small values of M_p/M_y, such as sections with hybrid webs, a relatively small deck area and a high-strength tension flange. It is somewhat less restrictive for sections with large values of M_p/M_y. Wittry (1993) considered various experimental test results and performed a large number of parametric cross-section analyses. The smallest experimental or theoretical resistance of all the cross-sections considered in this research and in other subsequent studies is $0.96M_p$. Eq. 6.10.7.1.2.2 is based on the target additional margin of safety of 1.28 specified by Wittry at the maximum allowed value of D_p combined with an assumed theoretical resistance of $0.96M_p$ at this limit. At the maximum allowed value of D_p specified by Eq. 6.10.7.3-1, the resulting nominal design flexural resistance is $0.78M_p$.~~

This page is intentionally left blank.

6.10.10.4.1—General

Replace Eq. (6.10.10.4.2-8) as follows:

$$\cancel{P_{2n} = 0.45 f'_c b_s t_s}$$

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

where:

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

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

This page is intentionally left blank.

6.10.11.1—Transverse Stiffeners

6.10.11.1.1—General

Revise the 2nd and 4th Paragraphs as follows:

Stiffeners ~~in straight girders~~ not used as connection plates shall be welded to ~~tight fit at~~ the compression flange and fitted tightly to the tension flange.~~, but need not be in bearing with the tension flange.~~ Single-sided stiffeners on horizontally curved girders should be attached to both flanges. ~~When pairs of transverse stiffeners are used on horizontally curved girders, they shall be fitted tightly to both flanges.~~

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 ~~but shall not exceed the lesser of~~ $6t_w$. In no case shall the distance exceed ~~and~~ 4.0 in.

This page is intentionally left blank.

6.10.11.2.1—General

Revise the 4th Paragraph as follows:

Each stiffener shall be ~~either milled~~ attached to ~~bear against~~ the flange through which it receives its load ~~or attached to that flange by a full penetration groove weld.~~ by one of the following:

- Milled or ground to bear plus fillet weld both sides,
- Full penetration groove weld.

This page is intentionally left blank.

6.13.1—General

Revise the 1st Paragraph as follows:

Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than ~~the larger of~~:

- ~~The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or~~
- 100 ~~75~~ percent of the factored ~~flexural, shear, or axial~~ resistances of the member or element.

This page is intentionally left blank.

6.13.6.1.4b—Web Splices

Revise the 2nd Paragraph as follows:

As a minimum, at the strength limit state, the design shear, V_{uw}, shall be taken as the smaller factored shear resistance of the girder webs at the point of splice, $\phi_v V_n$ ~~follows~~:

- ~~If $V_u < 0.5\phi_v V_n$, then:~~

$$\cancel{V_{uw} = 1.5V_u} \quad \cancel{(6.13.6.1.4b\text{-}1)}$$

- ~~Otherwise:~~

$$\cancel{V_{uw} = \frac{(V_u + \phi_v V_n)}{2}} \quad \cancel{(6.13.6.1.4b\text{-}2)}$$

$$\underline{V_{uw} = \phi_v V_n} \quad \underline{(6.13.6.1.4b\text{-}1)}$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

~~V_u = shear due to the factored loading at the point of splice (kip)~~

V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively (kip)

C6.13.6.1.4b

Delete the 1st Paragraph

~~Eqs. 6.13.6.1.4b-1 and 6.13.6.1.4b-2 provide a more consistent design shear to be used for designing web splice plates and their connections at the strength limit state than that given in past editions of the Standard Specifications and the First Edition of the LRFD Specifications. Eq. 6.13.6.1.4b-1 arbitrarily limits the increase in the shear at the point of splice to 50 percent of the shear due to the factored loading, V_u, where V_u is less than 50 percent of the factored shear resistance, $V_r = \phi_v V_n$, at the point of splice. The increase in the shear is limited to 50 percent of V_u because the possibilities for V_u to change from its calculated value are less than for moment; large unintended shifts in the shear at the splice are unlikely. In addition, the maximum shear is usually not concurrent with the maximum moment at the splice. Thus, the use of a lower value of the design shear in regions where the applied shear is low is deemed reasonable. A lower value of the design shear is also more reasonable for rolled beams, which have significantly higher values of factored shear resistance. For cases where V_u is greater than 50 percent of V_r, the design shear is determined from Eq. 6.13.6.1.4b-2 as the average of V_u and V_r. For checking slip of the bolted connections, the design shear is simply taken as the shear at the point of splice under Load Combination Service II defined in Table 3.4.1-1. The web with the smallest nominal shear resistance on either side of the splice should be used to determine the design shear.~~

This page is intentionally left blank.

C6.13.6.1.4b

Revise Eqs. (C6.13.6.1.4.b-1) and (C6.13.6.1.4b-2) as follows:

For compact sections:

$$M_{uw} = \phi_f \frac{t_w F_{yw}}{4}\left(D^2 - 4y_o^2\right) \quad \text{(C6.13.6.1.4b-1a)}$$

$$H_{uw} = \phi_f \left(2t_w y_o F_{yw}\right) \quad \text{(C6.13.6.1.4b-2a)}$$

For noncompact sections:

$$M_{uw} = \phi_f \frac{t_w D^2}{12}\left(F_{nc} + F_{yw}\right) \quad \text{(C6.13.6.1.4b-1b)}$$

$$H_{uw} = \phi_f \frac{t_w D}{2}\left(F_{yw} - F_{nc}\right) \quad \text{(C6.13.6.1.4b-2b)}$$

$$\cancel{M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right|} \quad \cancel{\text{(C6.13.6.1.4b-1)}}$$

$$\cancel{M_{uw} = \frac{t_w D}{2}\left(R_h F_{cf} + R_{cf} f_{ncf}\right)} \quad \cancel{\text{(C6.13.6.1.4b-2)}}$$

where:

t_w = web thickness of the smaller section at the point of splice (in.)

D = web depth of the smaller section at the point of splice (in.)

~~R_h = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

~~F_{cf} = design stress for the controlling flange at the point of splice specified in Article 6.13.6.1.4c; positive for tension, negative for compression (ksi)~~

~~R_{cf} = the absolute value of the ratio of F_{cf} to the maximum flexural stress, f_{cf}, due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in Article 6.13.6.1.4c~~

~~f_{ncf} = flexural stress due to the factored loads at the midthicknes of the noncontrolling flange at the point of splice concurrent with f_{cf}; positive for tension, negative for compression (ksi)~~

F_{nc} = nominal flexural resistance of the compression flange at the point of splice as specified in Article 6.10.8.2 (ksi)

F_{yw} = specified minimum yield strength of the web at the point of splice (ksi)

y_g = distance from the mid-depth of the web to the plastic neutral axis (in.)

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

Revise the 4th Paragraph as follows:

In Eqs. C6.13.6.1.4b-1a to C6.13.6.1.4b-2b, ~~C6.13.6.1.4b-1 and C6.13.6.1.4b-2,~~ it is suggested that M_{uw} and H_{uw} be computed by conservatively using the flexural resistance ~~stresses at the midthickness~~ of the compression flanges and specified minimum yield strength of the web. ~~By utilizing the stresses at the midthickness of the flanges, the same stress values can be used for the design of both the web and flange splices, which simplifies the calculations. As an alternate, however, the stresses at the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross-sections supporting those loadings. In Eqs. C6.13.6.1.4b-1 and C6.13.6.1.4b-2, the concurrent flexural stress at the midthickness of the noncontrolling flange is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. The controlling and noncontrolling flanges are defined in Article C6.13.6.1.4c.~~

Revise the 6th Paragraph as follows:

Eqs. C6.13.6.1.4b-1c and C6.13.6.1.4b-2c can ~~also~~ be used to compute values of M_{uw} and H_{uw} to be used when checking for slip of the web ~~bolts~~ connections. ~~However, the following substitutions must first be made in both equations:~~

- ~~Replace F_{cf} with the maximum flexural stress, f_s, due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of splice,~~
- ~~Rreplace f_{ncf} with the flexural stress, f_{os}, due to Load Combination Service II at the midthickness of the other flange at the point of splice concurrent with f_s in the flange under consideration, and~~
- ~~Set the factors R_h and R_{cf} equal to 1.0. It is not necessary to determine a controlling and noncontrolling flange when checking for slip. The same sign convention applies to the stresses.~~

$$M_{uw} = \frac{t_w D^2}{12}\left|f_s - f_{os}\right| \qquad \text{(C6.13.6.1.4b-1c)}$$

$$H_{uw} = \frac{t_w D}{2}\left(f_s + f_{os}\right) \qquad \text{(C6.13.6.1.4b-2c)}$$

where:

f_s = maximum flexural stress due to Load Combination Service II at the extreme 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 due to Load Combination Service II at the extreme fiber of the other flange of the smaller section at the point of splice with f_s in the flange under consideration (positive for tension and negative for compression) (ksi)

In Eqs. C6.13.6.1.4b-1c and C6.13.6.1.4b-2c, it is suggested that M_{uw} and H_{uw} be computed by conservatively using the stresses at the extreme fiber of the flanges. As an alternate, the stresses at the midthickness of the flanges or the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross-sections supporting those loadings.

This page is intentionally left blank.

6.13.6.1.4c—Flange Splices

Revise as follows:

At the strength limit state, splice plates and their connections on ~~the controlling~~ flanges shall be proportioned to provide a minimum resistance taken as the design stress, F_{cf}, times the smaller effective flange area, A_e, on either side of the splice, where F_{cf} is defined as:

$$\cancel{F_{cf} = \frac{\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}\right)}{2} \geq 0.75\alpha\phi_f F_{yf}}$$

$$\underline{F_{cf} = \alpha\phi_f F_{yf}} \qquad (6.13.6.1.4c\text{-}1)$$

in which:

A_e = effective area of the flange (in.^{2}). For compression flanges, A_e shall be taken as the gross area of the flange. For tension flanges, A_e shall be taken as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g \qquad (6.13.6.1.4c\text{-}2)$$

where:

~~f_{cf} = maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (ksi)~~

~~R_h = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

α = 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

F_n = nominal flexural resistance of the flange (ksi)

F_{yf} = specified minimum yield strength of the flange (ksi)

This page is intentionally left blank.

ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2
ϕ_y = resistance factor for yielding of tension members as specified in Article 6.5.4.2
A_n = net area of the tension flange determined as specified in Article 6.8.3 (in.^{2})
A_g = gross area of the tension flange (in.^{2})
F_u = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)
F_{yt} = specified minimum yield strength of the tension flange (ksi)

Delete the 2nd Paragraph

Delete Eq. (6.13.6.1.4c-3)

~~Splice plates and their connections on the noncontrolling flange at the strength limit state shall be proportioned to provide a minimum resistance taken as the design stress,~~ $\cancel{F_{ncf}}$, ~~times the smaller effective flange area,~~ $\cancel{A_e}$, ~~on either side of the splice, where~~ $\cancel{F_{ncf}}$ ~~is defined as:~~

$$\cancel{F_{cf} = R_{cf}\left|\frac{f_{ncf}}{R_h}\right| \geq 0.75\alpha\phi_f F_{yf}} \qquad \text{(6.13.6.1.4c-3)}$$

~~where:~~

$\cancel{R_{cf}}$ ~~= the absolute value of the ratio of~~ $\cancel{F_{cf}}$ ~~to~~ $\cancel{f_{cf}}$ ~~for the controlling flange~~
$\cancel{f_{ncf}}$ ~~= flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with~~ $\cancel{f_{cf}}$ ~~(ksi)~~
$\cancel{R_h}$ ~~= hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which~~ $\cancel{F_{cf}}$ ~~does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

C6.13.6.1.4c

Delete the 3rd Paragraph

~~The controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its midthickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the noncontrolling flange. In areas of stress reversal, the splice must be checked independently for both positive and negative flexure. For composite sections in positive flexure, the controlling flange is typically the bottom flange. For sections in negative flexure, either flange may qualify as the controlling flange.~~

Delete the 5th Paragraph.

~~Eq. 6.13.6.1.4c-3 defines a design stress for the noncontrolling flange at the strength limit state. In Eq. 6.13.6.1.4c-3, the flexural stress at the midthickness of the noncontrolling flange, concurrent with the stress in the controlling flange, is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. However, as a minimum, the factored-up stress must be equal to or greater than~~ $\cancel{0.75\alpha\phi_f F_{yf}}$.

Delete the 7th Paragraph.

~~Since flanges of hybrid sections are allowed to reach~~ $\cancel{F_{yf}}$, ~~the applied flexural stress at the midthickness of the flange in Eqs. 6.13.6.1.4c-1, 6.13.6.1.4c-3, and 6.13.6.1.4c-5 is divided by the hybrid factor,~~ $\cancel{R_h}$, ~~instead of reducing~~ $\cancel{F_{yf}}$ ~~by~~ $\cancel{R_h}$. ~~In actuality, yielding in the web results in an increase in the applied flange stress. When the flange design stress is less than or equal to the specified minimum yield strength of the web,~~ $\cancel{R_h}$ ~~is taken equal to 1.0 since there is theoretically no yielding in the web. The load shedding factor,~~ $\cancel{R_b}$, ~~is not included in these equations since the presence of the web splice plates precludes the possibility of local web buckling.~~

6.13.6.1.4c— Flange Splices

Revise f_s definition after Eq. (6.13.6.1.4c-5) as follows:

f_s = maximum flexural stress due to Load Combination Service II at the extreme fiber ~~midthickness~~ of the flange under consideration for the small section at the point of splice (ksi)

C6.13.6.1.4c

Revise the 10th Paragraph as follows:

For box section cited in this Article, including sections in horizontally curved bridges, longitudinal warping stresses due to cross-section distortion can be significant under construction and service conditions and must therefore be considered when checking the connections of bolted flange splices for slip for fatigue. The warping stresses in these cases can typically be ignored in checking the top-flange splices once the flange is continuously braced. The warping stresses can also be ignored when checking splices in both the top and bottom flanges at the strength limit state. For these sections, St. Venant torsional shear must also be considered in the design of box-flange bolted splices at all limit states. St. Venant torsional shears are typically neglected in top flanges of tub sections once the flanges are continuously braced. The bolts for box-flange splices may be designed for the effects of the torsional shear using the traditional elastic vector method that is typically applied in the design of web splices. Depending on the limit state under investigation, the shear on the flange bolt group is assumed caused by either the flange force due to the factored loads, or by the appropriate flange design force, as applicable. The moment on the bolt group is taken as the moment resulting from the eccentricity of the St. Venant torsional shear due to the factored loads, assumed applied at the centerline of the splice. At the strength limit state, the torsional shear due to factored loads should be used. ~~need not be multiplied by the factor,~~ ~~R_{cf}, from Eq. 6.13.6.1.4c-3 when computing the moment for the design of the splice.~~ The box-flange splice plates in these cases should also be designed at the strength limit state for the combined effects of the appropriate flange force and the moment resulting from the eccentricity of the St. Venant torsional shear due to the factored loads.

C6.13.6.1.4c

Revise the 11th Paragraph as follows:

In cases for straight girders where flange lateral bending is deemed significant, and for horizontally curved girders, the effects of the lateral bending must be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections. The traditional elastic vector method may also be used in these cases to account for the effects of flange lateral bending on the design of the splice bolts. The shear on the flange bolt group is assumed caused by the flange force, calculated as described in the preceding paragraph. The flange force is calculated without consideration of the flange lateral bending. The moment on the bolt group is taken as the flange lateral bending moment due to the factored loads. At the strength limit state, the flange lateral bending moment due to the factored loads should be used. ~~need not be multiplied by the factor, R_{cf}, from Eq. 6.13.6.1.4c-3 when computing the moment for the design of the splice.~~ Splice plates subject to flange lateral bending should also be designed at the strength limit state for the combined effects of the appropriate flange force and the flange lateral bending moment due to the factored loads. Lateral flange bending can be ignored in the design of top flange splices once the flange is continuously braced.

6.13.6.2—Welded Splices

Revise the 2nd Paragraph as follows:

Welded splices shall be designed ~~to resist the design moment, shear, or axial force specified in Article 6.13.1.~~ at the strength limit state for not less than 100 percent of the factored resistances of the member or element. Tension and compression members may be spliced by means of full penetration butt welds; splice plates should be avoided.

Revise the 3rd Paragraph as follows:

~~Welded field splices should be arranged tom minimize overhead welding.~~ Splices, except for orthotropic decks. shall not be field welded.

6.14.2.8—Gusset Plates

Revise as follows:

6.14.2.8.1—General

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.

The nominal resistance of a gusset plate shall be based on the effective width as shown in Figure C6.14.2.8.1-1. The gross and net cross-section area shall be on the effective width.

The provisions of Articles 6.13.2, 6.13.3, 6.13.4 and 6.13.5 shall apply, as applicable.

Gusset or connection plates should be used for connecting main members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration.

Re-entrant cuts, except curves made for appearance, should be avoided as far as practicable.

~~The maximum stress from combined factored flexural and axial loads shall not exceed $\phi_f F_y$ based on the gross area.~~

~~The maximum shear stress on a section due to the factored loads shall be $\phi_v F_u/\sqrt{3}$ for uniform shear and $\phi_v 0.74 F_u/\sqrt{3}$ for flexural shear computed as the factored shear force divided by the shear area.~~

~~If the length of the unsupported edge of a gusset plate exceeds $2.06(E/F_y)^{1/2}$ times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.~~

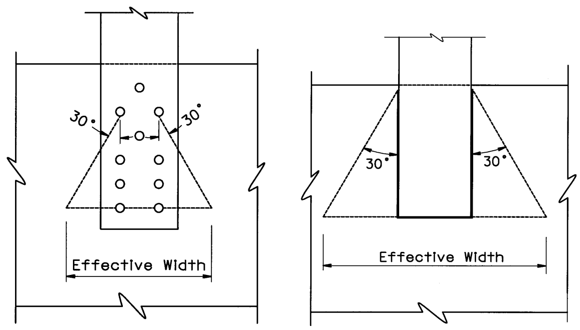
C6.14.2.8

Revise as follows:

C6.14.2.8.1

Major revisions are based on Caltrans successful practice and Caltrans *Guide Specifications for Seismic Design of Steel Bridges* (*Caltrans 2001).*

Figure C6.14.2.8.1-1 shows the effective width for a gusset plate in accordance with Whitmore's method (Whitmore 1952).



(a) Bolted Gusset Plate (b) Welded Gusset Plate

Figure C6.14.2.8.1-1—Effective Width of Gusset Plate

6.14.2.8.2—Limiting Unsupported Edge Length to Thickness Ratio

The unsupported edge length to thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \leq 2.06\sqrt{\frac{E}{F_y}} \qquad (6.14.2.8.2\text{-}1)$$

where:

L_g = unsupported edge length of a gusset plate (in.)
t = thickness of a gusset plate (in.)
E = modulus of elasticity of steel (ksi)
F_y = specified minimum yield strength of the gusset plate (ksi)

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

$$I_s \geq \begin{cases} 1.83\,t^4\sqrt{(b/t)^2 - 144} \\ 9.2\,t^4 \end{cases} \qquad (6.14.2.8.2\text{-}2)$$

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.)

C6.14.2.8.2

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*)

6.14.2.8.3—Tensile Resistance

The tensile resistance of a gusset plate shall be:

$$\phi P_n = \phi_y A_g F_y \leq \begin{cases} \phi_u A_n F_u \\ R_r \end{cases} \qquad (6.14.2.8.3\text{-}1)$$

where:

A_n = net cross-section area of a gusset plate (in.^{2})
A_g = gross cross-section area of a gusset plate (in.^{2})
C6.14.2.8.2

C6.14.2.8.3

This requirement is to ensure that the tensile strength is governed by yielding in the gross section, and fracture in the net section and block shear rupture are prevented.

F_u = specified minimum tensile strength of the gusset plate (ksi)

ϕ_u = resistance factor for tension fracture in net section = 0.80

ϕ_y = resistance factor for tension yielding in gross section = 0.95

R_r = factored block shear rupture resistance specified by Article 6.13.4

6.14.2.8.4—Compressive Resistance

The nominal compressive resistance of a gusset plate, P_n, shall be calculated in accordance with Article 6.9.4.1.

C6.14.2.8.4

The effective length factor, *K* in Eqs. (6.9.4.1-1) and (6.9.4.1-2) may be taken as 0.6 for the gusset supported by both edges, and 1.2 for the gusset supported by one edge only (*AISC 2001*); A_s is the average effective cross section area defined by Whitmore's method; *l* is the perpendicular distance from the Whitmore section to the interior corner of the gusset. For members that are not perpendicular to each other as shown in Figure C6.14.8.2.4-1 (*AISC 2001*), *l* can be alternatively determined as the average value of

$$l = \frac{L_1 + L_2 + L_3}{3} \qquad \text{(C6.14.2.8.4-1)}$$

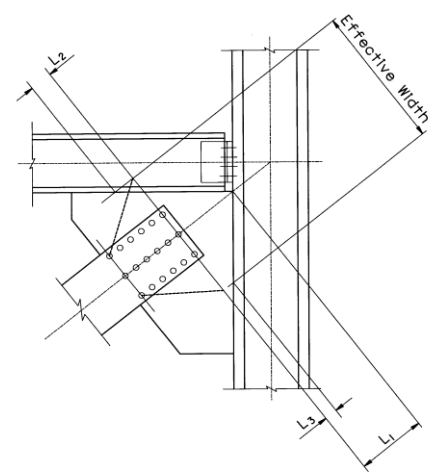


Figure C6.14.2.8.4-1—Gusset Plate Connection

where:

L_1 = distance from the centerline of the Whitmore section to the interior corner of a gusset plate (in.)

L_2, L_3 = distance from the outside corner of the Whitmore section to the edge of a member; negative value shall be used when the part of Whitmore section enters into the member (in.)

6.14.2.8.5—Flexural Resistance

The nominal flexural resistance of a gusset plate, M_n, shall be determined by:

$$M_n = S F_y \quad (6.14.2.8.5\text{-}1)$$

where:

S = elastic section modulus of the cross section of a gusset plate (in.^{3})

6.14.2.8.6—Shear Resistance

The nominal shear resistance of a gusset plate, V_n, shall be determined by:

$$V_n = 0.58 F_y A_g \quad (6.14.2.8.6\text{-}1)$$

where:

A_g = gross cross-section area of a gusset plate (in.^{2})

6.14.2.8.7—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:

$$\frac{M_{ux}}{\phi_f S_x F_y} + \frac{M_{uy}}{\phi_f S_y F_y} + \frac{P_u}{\phi F_y A_g} \leq 1 \quad (6.14.2.8.7\text{-}1)$$

where:

ϕ_f = resistance factor for flexural
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)
S_x = elastic section modulus about x-x axis of the gusset plate (in.^{3})
S_y = elastic section modulus about y-y axis of the gusset plate (in.^{3})
A_g = gross area of the gusset plate (in.^{2})
F_y = specified minimum yield strength of the gusset plates (ksi)

6.14.2.8.8—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.

For single gusset plate connections, out-of-plane moment and shear are about the weak axis.

This page is intentionally left blank.

6.17—REFERENCES

Add the following References:

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

Caltrans, 2001. *Guide Specifications for Seismic Design of Steel Bridges*, First Edition, California Department of Transportation, Sacramento, CA.

Whitmore, R. E. 1952. "Experimental Investigation of Stresses in Gusset Plates," *Bulletin 16*, University of Tennessee, Knoxville, TN.

This page is intentionally left blank.

9.5.2—Service Limit States

Add a new last sentence to the 1st Paragraph as follows:

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 5.7.3.4.

This page intentionally left blank

9.7.1.1—Minimum Deck Thickness ~~Depth~~ and Cover

Revise as follows:

Unless approved by the Owner, the minimum thickness of the ~~depth of a~~ concrete deck, excluding any provision for grinding, grooving, and sacrificial surface, should conform to the deck design standards developed by Caltrans ~~not be less than 7.0 in~~.

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 5.12.3

C9.7.1.1

Revise the 3rd Paragraph as follows:

The combinations of minimum concrete cover, concrete mix design and the need for protective coatings on reinforcement described in Article 5.12.3 are based on the results of monitoring bridges in California. ~~Minimum cover requirements are based on traditional concrete mixes and on the absence of protective coating on either the concrete or steel inside. A combination of special mix design, protective coatings, dry or moderate climate, and the absence of corrosion chemicals may justify a reduction of these requirements provided that the Owner approves~~.

9.7.1.4–Edge Support

Revise the 2nd Paragraph as follows:

Where the primary direction of the deck is transverse, ~~and/or the deck is composite with a structurally continuous concrete barrier,~~ no additional edge beam need be provided.

9.7.2.2—Application

Revise the 1st Paragraph as follows:

Empirical design of reinforced concrete decks and overhangs shall ~~may~~ not be used ~~be used if the conditions set forth in Article 9.7.2.4 are satisfied~~.

Remove the 2nd Paragraph

~~The provisions of this Article shall not be applied to overhangs.~~

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.

This page intentionally left blank

10.3—NOTATION

Revise as follows:

φ_{qp} = resistance factor for tip resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5) (10.9.3.5.1)
φ_{qs} = resistance factor for ~~shaft~~ side resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5)

This page is intentionally left blank.

10.5.2.1—General

Revise the 1st Paragraph as follows:

Foundation design at the service limit state shall include:

- Settlements,
- Horizontal movements,
- Overall stability, and
- Total s~~S~~cour at the design flood.

10.5.2.2—Tolerable Movements and Movement Criteria

Add two paragraphs after the 3rd Paragraph:

Limit eccentricity under Service-I load combination 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 the following after the last paragraph:

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

Revise the 2nd Paragraph as follows:

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

- Structural resistance and
- Loss of lateral and ~~vertical~~ axial support due to scour at the design flood event.

C10.5.3.1

Revise the 4th Paragraph as follows:

The design ~~event~~ flood for scour is defined in ~~Section 2~~ Article 2.6 and is specified in Article 3.7.5 as applicable at the strength limit state.

This page is intentionally left blank.

C10.5.4.1

Revise the 1st Paragraph as follows:

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. Appendix A10 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.

This page is intentionally left blank.

10.5.5.2.1 —General

Revise as follows:

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~~, unless regionally specific values or substantial successful experience is available to justify higher values~~.

C10.5.5.2.1

Revise as follows:

~~Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values.~~ 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 ~~sand or clay~~. Many ~~N~~naturally occurring soils do not fall neatly into these two classifications. In general, the terms ~~"sand" and~~ "cohesionless soil" or “sand” may be connoted to mean drained conditions during loading, while ~~"clay" or~~ "cohesive soil" or “clay” implies undrained conditions in the short-term. For other or intermediate soil classifications, such as clayey sand or silts ~~or gravels~~, 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 ~~or~~, undrained, or a combination of the two strengths ~~strength,~~ 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). ~~If the resistance factors provided in this article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system.~~

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 unit is usually more, and in many cases considerably

This page is intentionally left blank.

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. ~~AASHTO Standard Specifications for Highway Bridges (2002).~~

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).

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

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

Revise as follows:

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings~~, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2~~.

Revise Table 10.5.5.2.2-1 as follows:

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/~~Soil/~~Condition		Resistance Factor
Bearing ~~Resistance~~ in Compression	φ_b	Theoretical method (Munfakh et al., 2001), in ~~clay~~ cohesive soils	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, 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_τ	Precast concrete placed on sand	0.90
		Cast-in-place concrete on sand	0.80
		Cast-in-place or pre-cast concrete on clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

This page is intentionally left blank.

10.5.5.2.3— Driven Piles

Delete the entire article and replace with the following:

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

C10.5.5.2.3

Delete the entire commentary and replace with the following:

The resistance factors in Table 10.5.5.2.3-1 are based on engineering judgment, and past WSD and Load Factored Design (LFD) practices.

This page is intentionally left blank.

Replace Table 10.5.5.2.3-1 as follows:

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 5.5.4.2.1
	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 5.5.4.2.1	
	Timber Piles	See the provisions of Articles 8.5.2.2 and 8.5.2.3	

This page is intentionally left blank.

10.5.5.2.4—Drilled Shafts

Delete the entire article and replace with the following:

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

C10.5.5.2.4

Delete the entire commentary and replace with the following:

The resistance factors in Table 10.5.5.2.4-1 are based on engineering judgment, and past WSD and LFD practices.

The maximum value of the resistance factors in Table 10.5.5.2.4-1 are based on an assumed normal level of field quality control during shaft construction. If a normal level of 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.

This page is intentionally left blank.

Replace Table with the following:

Table 10.5.5.2.4-1—Geotechnical 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

Revise the 1st Paragraph as follows:

~~The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods.~~ <u>See Commentary to Article 3.4.1, Extreme Events, and Article 3.7.5.</u>

10.5.5.3.3—Other Extreme Event Limit States

Revise the 1st Paragraph as follows:

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. ~~For the uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.~~

C10.5.5.3.3

Delete the entire commentary:

~~The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see ArticleC10.5.5.2.3).~~

10.6.1.1—General

Revise the 1st Paragraph as follows:

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

Revise the commentary as follows:

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

Revise as follows:

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.

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

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

$$L' = L\text{-}\underline{2}e_L$$

Where,

e_B = $\underline{M_L/V}$ $\underline{=}$ Eccentricity parallel to dimension *B* (ft)

e_L = $\underline{M_B/V}$ $\underline{=}$ Eccentricity parallel to dimension *L* (ft)

$\underline{M_B}$ $\underline{=}$ Factored moment about the central axis along dimension *B* (kip-ft)

$\underline{M_L}$ $\underline{=}$ Factored moment about the central axial along dimension *L* (kip-ft)

$\underline{V}$ $\underline{=}$ Factored vertical load (kips)

10.6.1.4—Bearing Stress Distributions

Revise 1st Paragraph as follows:

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

- Uniform over the effective area for footing on soils, or

C10.6.1.3

Add the following reference:

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

10.6.1.6—Groundwater

Modify the last paragraph as follows:

The influences of groundwater ~~table~~ 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.

This page is intentionally left blank.

C10.6.2.4.1

Insert 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

Revise the 3rd Paragraph as follows:

The elastic half-space method assumes the footing ~~is flexible and~~ 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:

C10.6.2.4.2

Modify the 6th Paragraph as follows:

~~The stress distribution used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth.~~ 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.

10.6.2.4.2—Settlement of Footing on Cohesionless Soils

Revise the last sentence in the last paragraph as follows:

In Figure 10.6.2.4.2-1, $\underline{N1}$ ~~N'~~ shall be taken as $\underline{(N_1)_{60}}$ ~~$N1_{60}$~~, Standard Penetration Resistance, N (blows/ft), corrected for <u>hammer energy efficiency and</u> overburden pressure as specified in Article 10.4.6.2.4.

C10.6.2.4.2

Modify the last sentence of the 8th Paragraph as follows:

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 ~~footings~~ <u>foundations</u>.

10.6.2.4.3—Settlement of Footings on Cohesive Soils

Insert 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.

10.6.2.4.3—Settlement of Footings on Cohesive Soils

Insert 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.

This page is intentionally left blank.

C10.6.3.1.2e

Replace H with H_{s2} in Eqs. C10.6.3.1.2e-5 and C10.6.3.1.2e-6 of commentary.

Revise equations as follows:

- For circular or square footings:

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

$$N_c^* = 6.17$$

- For strip footings:

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

$$N_c^* = 5.14$$

This page is intentionally left blank.

10.6.3.1.2e—Two-Layered Soil System in Undrained Loading

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

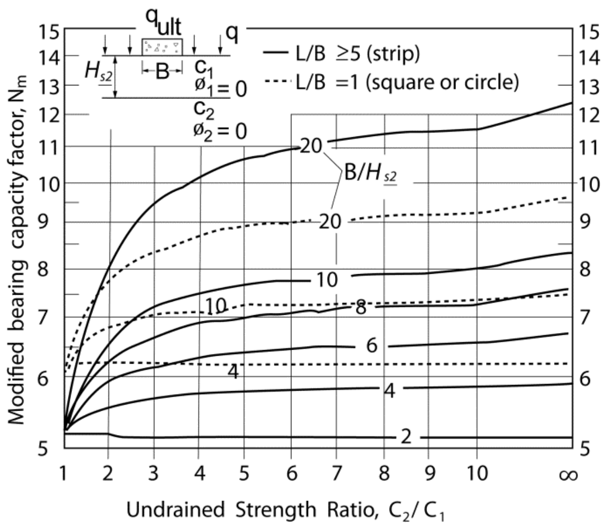


Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f —Two-Layered Soil System in Drained Loading

Replace H with H_{s2} in Eq. 10.6.3.1.2f-1.

Revise equation as follows:

$$\cancel{q_n = \left[q_2 + \left(\frac{1}{K}\right) c_1' \cot\varphi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\varphi_1' \left(\frac{H}{B}\right)} - \left(\frac{1}{K}\right) c_1' \cot\varphi_1'}$$

$$\underline{q_n = \left[q_2 + \left(\frac{1}{K}\right) c_1' \cot\varphi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\varphi_1' \left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right) c_1' \cot\varphi_1'} \qquad (10.6.3.1.2f\text{-}1)$$

C10.6.3.1.2f

Replace H with H_{s2} in Eq. C10.6.3.1.2f-1 of the commentary.

Revise equation as follows:

$$\cancel{q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H}{B}}}$$

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

Revise title as follows:

10.6.3.1.3—Semiempirical Procedures <u>for Cohesionless Soils</u>

C10.6.3.1.3

Add the following to the end of article:

<u>It is recommended that the *SPT* based method not be used.</u>

C10.6.3.2.1

Revise as follows:

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 ~~strength~~ limit state before checking nominal bearing resistance at both ~~the service and~~ strength and extreme event limit states.

Revise title as follows:

10.6.3.2.4—Plate Load Test

Revise as follows:

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

Revise as follows:

~~The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:~~

- ~~One third of the corresponding footing dimension, *B* or *L*, for footings on soils, or 0.45 of the corresponding footing dimensions *B* or *L*, for footings on rock.~~

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

Revise as follows:

~~A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of *B*/3 were comparable to those of ASD with an eccentricity of *B*/6. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of *B*/2 would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.~~

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 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

10.6.3.4—Failure by Sliding

Revise Figure 10.6.3.4-1 as follows:

Replace Q_τ with R_τ

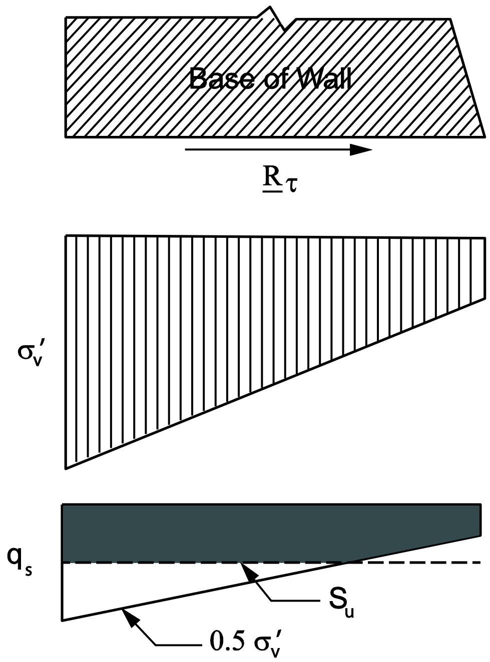


Figure 10.6.3.4-1—Procedure for Estimating Nominal Sliding Resistance for Walls on Clay

This page is intentionally left blank.

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

Revise the 1st Paragraph as follows:

Center-to-center spacing should not be less than ~~30.0 in. or 2.5~~ 36.0 in. and 2.0 pile diameters. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in. and 0.5 pile diameters.

Revise the 2nd Paragraph as follows:

~~The tops of piles shall project at least 12.0 in. into the pile cap after all damaged material has been removed.~~ If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than ~~6.0 in.~~ 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:

In general, battered piles should not be used for foundations of bents and piers.

10. 7.1.5—Pile Design Requirements

Revise as follows:

Pile design shall address the following issues as appropriate:

- Pile cut off elevation, ~~Nominal bearing resistance to be specified in the contract,~~ 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 from estimated pile penetration ~~required~~ to meet nominal axial resistance and other design requirements.
- ~~Minimum pile penetration necessary to satisfy the requirements caused by u~~Uplift, lateral loads, scour, downdrag, ~~settlement,~~ liquefaction, lateral spreading ~~loads~~, 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.

C10.7.1.6.2

Revise the 1st and 2nd Paragraphs as follows:

Static downdrag does not affect the ultimate geotechnical capacity or nominal resistance of the pile foundations. It acts to increase pile settlement, and the load on the pile or pile group and the cap. ~~Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See commentary to Article C3.11.8.~~

In the case of friction piles with limited tip resistance, the downdrag ~~load~~ can ~~exceed the geotechnical resistance of the pile,~~ cause~~ing~~ the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by nominal bearing resistance below the downdrag zone, service limit state tolerances ~~will~~ may govern the geotechnical design.

10.7.2.2—Tolerable Movements

Revise as follows:

The provisions of Article 10.5.2.~~1~~ 2. shall apply.

10.7.2.3—Settlement

C10.7.2.2

Revise as follows:

See Article C10.5.2.~~1~~ 2.

C10.7.2.3

Add the following:

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.

Revise title as follows:

10.7.2.3.2 Pile Group~~s~~ Settlement ~~in Cohesive Soil~~

Revise the 1st Paragraph as follows:

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 ~~10.7.2.3.1-1.1~~ 10.7.2.3.1-1 or Figure 10.7.2.3.1-2.

Revise the 2nd Paragraph as follows:

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

q = net foundation pressure ~~applied at $2D_b/3$ as shown in Figure 10.7.2.3.1-1~~; 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)

D_b = depth of embedment of piles in the cohesionless layer that provides support~~, as specified in Figure 10.7.2.3.1-1~~ (ft)

Revise the 4th Paragraph as follows:

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. ~~The *SPT* and *CPT* methods (Eqs. 10.7.2.3.2-1 and 10.7.2.3.2-2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1-1b and Figure 10.7.2.3.1-2.~~

This page is intentionally left blank.

10.7.2.4—Horizontal Pile Foundation Movement

Revise Table as follows:

Table 10.7.2.4-1 Pile P-Multipliers, P_m for Multiple Row Shading ~~(average from Hannigan et al., 2005).~~

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 ~~0.8~~	0.55 ~~0.4~~	0.40 ~~0.3~~
5.0*B*	1.0	0.85	0.70
7.0*B*	1.0	1.0	0.90

Revise the 7th Paragraph as follows:

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 ~~group reduction factor~~ of less than 1.0 shall only be used if the pile spacing is ~~5~~4*B* or less~~, i.e., a *Pm* of 0.8 for a spacing of 3*B*, as shown in Figure 10.7.2.4-1~~. 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

Revise the 6th Paragraph as follows:

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

This page is intentionally left blank.

10.7.2.5—Settlement Due to Downdrag

Delete the 1st and 2nd Paragraphs and add the following:

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

C10.7.2.5—Settlement Due to Downdrag

Delete the 1st and 2nd Paragraphs and add 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

Revise as follows:

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 ~~Estimated pile length~~ 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 ~~minimum pile penetration required, if applicable~~, 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 ~~achieve the required nominal axial resistance or minimum penetration~~ 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

Revise the 1st Paragraph as follows:

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. ~~Assuming static load tests, dynamic methods, e.g., dynamic test with signal matching, wave equation, pile formulae, etc., are used during pile installation to establish when the nominal bearing resistance has been met, a minimum pile penetration should not be used to ensure that the required nominal pile bearing, i.e., compression, resistance is obtained.~~

This page is intentionally left blank.

Revise the title as follows:

10.7.3.3—Pile Length Estimates ~~for Contract Documents~~

Revise as follows:

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 for establishment of contract pile quantities. Local experience shall also be considered when making pile quantity estimates, both to select an estimation method and to assess the potential prediction bias for the method used to account for any tendency to over-predict or under-predict pile compressive resistance. 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 <u>the specified tip elevation and</u> estimating contract pile quantities.

C10.7.3.3

Revise the 1st and 2nd Paragraphs as follows:

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 ~~required~~ <u>specified</u> pile <u>tip elevation</u> ~~or length~~ 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 probe piles across the site ~~is often~~ <u>may be</u> used to determine variable pile order lengths. Probe piles are particularly useful when driving concrete piles. <u>The specified pile tip elevation</u> ~~or length~~ 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.

Revise the 5th Paragraph as follows:

~~The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience.~~ 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~~, and the Eq. C10.7.3.3-1 is likely not applicable~~.

Delete the 6th Paragraph.

Delete the 7th Paragraph.

This page is intentionally left blank.

C10.7.3.4.3

Revise the 3rd Paragraph as follows:

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). ~~Furthermore, the resistance factors provided in Table 10.5.5.2.3-1 for driving formulas were developed for end of driving conditions and empirically have been developed based on the assumption that soil setup will occur. See Article C10.5.5.2.3 for additional discussion on this issue.~~

C10.7.3.6

Revise the 1st Paragraph as follows:

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.

Revise the 2nd Paragraph as follows:

The magnitude of skin friction that will be lost due to scour may be estimated by static analysis. 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. D~~ 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.

C10.7.3.7

Add the following at the end of the article:

Additional guidance to estimate downdrag on single pile and pile groups are provided in ASCE (1993), Briaud and Tucker (1997), and Hennigan et al. (2005).

10.7.3.8.1—General

Revised as follows:

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 ~~based on the method used to verify pile axial resistance~~ 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. ~~and, if required, to a minimum penetration needed for uplift, scour, lateral resistance, or other requirements as specified in Article 10.7.6. If it is determined that static load testing is not feasible and dynamic methods are unsuitable for field verification of nominal bearing resistance, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in Article 10.7.6.~~

C10.7.3.8.1

Revise as follows:

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. ~~From this design step, the number of piles and pile resistance needed to resist the factored loads applied to the foundation are determined.~~ 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.

~~In cases where the project is small and the time to achieve soil setup is large compared with the production time to install all the piles, no field testing for the verification of nominal resistance may be acceptable.~~

10.7.3.8.2—Static Load Test

Revise the 1st Paragraph as follows:

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed ~~less than 5 days after the test pile was driven unless approved by the Engineer.~~ 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

Revise the Figure C10.7.3.8.2-1 as follows:

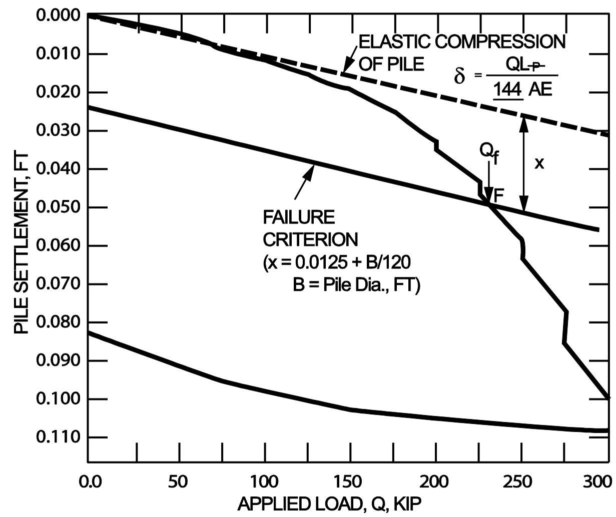


Figure C10.7.3.8.2-1 Davissons' ~~Alternate~~ Method for Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972)

10.7.3.8.3—Dynamic Testing

Revise the 1st Paragraph as follows:

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.

Add the following to the end of the article:

Dynamic testing shall not be used without calibrating to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.3

Revise the 1st Paragraph as follows:

The dynamic test may be used to establish the driving criteria at the beginning of production driving. The minimum number of piles that should be tested are as specified by the Engineer. A signal matching analysis (Rasusche 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.

10.7.3.8.4—Wave Equation Analysis

Add the following to the end of the article:

The wave equation shall not be used without calibrating to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.4

Revise the 1st Paragraph as follows:

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 pile bearing static 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

Revise the 1st Paragraph as follows:

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

$$\cancel{R_{ndr} = 1.75\sqrt{E_d}\log_{10}(10N_b) - 100}$$

$$R_{ndr} = [1.83*(E_r)^{1/2}*\log_{10}(0.83*N_b)]-124 \quad (10.7.3.8.5\text{-}1)$$

where:

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

~~E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-lb)~~

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

N_b = Number of hammer blows in the last foot, (maximum value to be used for N is 96) ~~for 1.0 in. of pile permanent set~~ (blows/~~in.~~ ft).

Delete the 2nd and 3rd Paragraphs.

Revise the 5th Paragraph as follows:

Dynamic formula should not be used when the required nominal resistance exceeds 600 kips or the pile diameter is greater than or equal to 18-in.

C10.7.3.8.5

Delete the 2nd Paragraph as follows:

~~Two dynamic formulas are provided here for the Engineer. If a dynamic formula is used, the FHWA Modified Gates Formula is preferred over the Engineering News Formula. It is discussed further in the Design and Construction of Driven Pile Foundations *(Hannigan et al., 2006)*. Note that the units in the FHWA Gates formula are not consistent. The specified units in Eq. 10.7.3.8.5-1 must be used.~~

Delete the 3rd Paragraph.

~~The Engineering News formula in its traditional form contains a factor of safety of 6.0. For LRFD applications, to produce a nominal resistance, the factor of safety has been removed. As is true of the FHWA Gates formula, the units specified in Eq. 10.7.3.8.5-2 must be used for the Engineering News formula. See Allen (2005, 2007) for additional discussion on the development of the Engineering News formulas and its modification to produce a nominal resistance.~~

Revise the 5th Paragraph as follows:

As the required nominal bearing resistance increases, the reliability of dynamic formulae tends to decrease. The modified ~~FHWA~~ Gates Formula tends to underpredict pile nominal resistance at higher resistances. ~~The Engineering News Formula tends to become unconservative as the nominal pile resistance increases. If other driving formulae are used, the limitation on the maximum driving resistance to be used should be based upon the limits for which the data is considered reliable, and any tendency of the formula to over or under predict pile nominal resistance.~~

C10.7.3.8.6a

Revise as follows:

~~While the most common use of static analysis methods is solely' for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal bearing resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that re-strike testing would require an impractical wait period by the Contractor on the site, e.g., soft silts or clays where a large amount of setup is anticipated.~~

The static analysis methods presented in this article should 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 should be used.

~~For use of static analysis methods for contract pile quantity estimation, see Article 10.7.3.3.~~

For open ended pipe piles, the nominal axial resistances should be calculated for both plugged and unplugged conditions. The lower of the two nominal resistances should be used for design.

10.7.3.10—Uplift Resistance of Single Piles

Revise the 1st and 2nd Paragraphs as follows:

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.

The nominal uplift resistance of a single pile should be estimated in a manner similar to that for estimating the side 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.

Revise the 5th Paragraph as follows:

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 load test results~~.~~, when available.

C10.7.3.10

Add before the 1st Paragraph as follows:

In general, piles may be considered to resist an intermittent or temporary, but not sustained, uplift by side friction.

Revise the 2nd Paragraph as follows:

~~Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static side friction resistance. Therefore, the side friction resistance estimated based on Article 10.7.3.8.6 does not need to be reduced to account for uplift effects on side friction.~~

See Hannigan et al. (2005) for guidance on the reduction of side friction due to the effects of uplift.

This page is intentionally left blank.

10.7.3.11—Uplift Resistance of Pile Groups

Revise the 4th Paragraph as follows:

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of $1H$ in $4V$ 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:

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.

This page is intentionally left blank.

Revise title as follows:

10.7.5 —Protection Against Corrosion and Deterioration

C10.7.5

Revise the 2nd Paragraph as follows:

As a minimum, the following types of deterioration shall be considered:

- Corrosion of steel pile foundations, particularly in fill soils,~~ low pH soil~~ and marine environment;
- Chloride, ~~S~~sulfate~~,~~ ~~chloride,~~ and acid attack of concrete pile foundations; and
- Decay of timber piles from wetting and drying cycles or from insects or marine borers.

Revise the 3rd Paragraph as follows:

The following soil, water or site conditions ~~should~~ shall be considered as ~~indicative~~ indicators of ~~a~~ potential pile corrosion or deterioration ~~or corrosion situation~~:

- Minimum ~~R~~resistivity equal to or less than ~~2,000~~ 1,000 ohm-cm,
- Chloride concentration equal to or greater than 500 ppm,
- Sulfate concentration equal to or greater than 2,000 ppm,
- pH equal to or less than 5.5,
- ~~pH between 5.5 and 8.5 in soils with high organic content,~~
- ~~Sulfate concentration greater than 1,000 ppm,~~
- Landfills and cinder fills,
- ~~Soils subject to m~~Mine or industrial drainage,
- Suspected chemical wastes, and
- Stray currents
- ~~Areas with a mixture of high resistivity soils and low resistivity high alkaline soils, and~~
- ~~Insects (woof piles)~~

Add the following after the 3rd Paragraph:

Steel piling may be used in corrosive soil and/or water environments provided the following corrosion rates are used to determine a corrosion allowance (sacrificial metal loss):

- 0.001 in. per year for the soil embedment zone,
- 0.004 in. per year for the immersed zone,
- 0.005 in. per year for the splash zone.

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 flange that would be exposed to the corrosive environment.

Delete the 4th Paragraph.

Revise the 12th Paragraph as follows:

Epoxy coating of pile reinforcement ~~has been found in some cases to be~~ is useful in resisting corrosion. It is important ~~to ensure~~ that the coating is continuous and free of holidays.

10.8.1.2—Shaft Spacing, Clearance, and Embedment Into Cap

Modify the 1st Paragraph as follows:

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 interaction effects between adjacent shafts shall be considered. If the center-to-center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents.~~

C10.8.1.2

Revise the 1st Paragraph as follows:

Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement. If the center-to-center spacing of drilled shafts is less than 3.0 diameters, the sequence of shaft installation should be specified in the contract documents.

Revise title as follows:

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

Revise as follows:

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 CIDH piles or drilled shafts, the minimum concrete cover to reinforcement (including epoxy coated rebar) shall be as specified in Table 10.8.1.3-1.

Table 10.8.1.3-1—Minimum Concrete Cover for CIDH Piles or Drilled Shafts (to be shown on the plan)

Diameter of the CIDH Pile or Drilled Shaft "*D*"	**Concrete Cover[a]**
16″ and 24″ Standard Plan Piles	Refer to the applicable Standard Plans
$24'' \leq D \leq 36''$	3″
$42'' \leq D \leq 54''$	4″
$60'' \leq D < 96''$	5″
96″ and larger	6″

[a] For shaft capacity calculations, only 3″ of cover is assumed effective and shall be used in calculations.

In order to improve concrete flow when constructing drilled shafts, a 5 in. × 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 longitudinal reinforcing spacing may be reduced from 5 in. to 3 in.

The maximum center-to-center spacing of longitudinal bars in drilled shafts is limited to 10 in. when the shaft diameter is less than 5 ft and 12 in. for larger shafts. The maximum center-to-center spacing of transverse bars in drilled shafts is limited to 8 in.

When a column is supported on a single enlarged Type II shaft (Caltrans' Seismic Design Criteria 2.2.4), the allowable offset between centerlines of the column (column cage centerline is fixed) and the shaft reinforcement cages shall be limited by the required horizontal clearance between the two cages. The clear distance between the two cages shall be at least 3.5 in. for dry pour and 5 in. for wet pour as shown in Figure 10.8.1.3-1. The offset between centerlines of the shaft cage and the drilled hole, shall be limited to provide minimum concrete cover of 3 in.

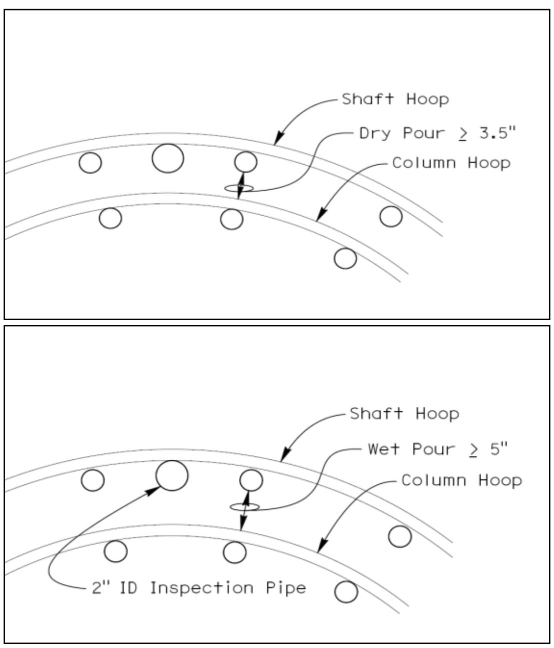


Figure 10.8.1.3-1—Clearance between Column and Shaft Rebar Cages in Enlarged Type II-Shafts

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.

10.8.2.2.2—Settlement of Single-Drilled Shaft

C10.8.2.2.2

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.

This page is intentionally left blank.

10.8.3.5.1c—Tip Resistance

Revise the 1st Paragraph.

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 O'Neil and Reese (1999) shall be ~~taken~~ calculated as follows:

If $Z \geq 3D$

~~$q_p = N_c S_u \leq 80.0$~~

$$q_p = N_c * S_u \qquad (10.8.3.5.1c\text{-}1)$$

in which:

~~$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$~~

$N_c = 9$ for $S_u \geq 2$ ksf

$$N_c = \left(\frac{4}{3}\right)\left[\ln\left(I_r\right) + 1\right] \text{ for } S_u < 2 \text{ ksf} \qquad (10.8.3.5.1c\text{-}2)$$

If $Z < 3D$,

$$q_p = \left(\frac{2}{3}\right)\left[1 + \left(\frac{1}{6}\right)\left(\frac{D}{B}\right)\right] N_c * S_u \qquad (10.8.3.5.1c\text{-}3)$$

where:

D = diameter of drilled shaft (ft)

Z = ~~penetration~~ depth of drilled shaft base (ft)

S_u = design undrained shear strength (ksf)

I_r = rigidity index = $(E_s/3S_u)$

E_s = Young's modulus of soil for undrained loading (ksf)

This page is intentionally left blank.

10.8.3.6.3—Cohesionless Soil

Revise Table as follows:

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*D*~~ 2.5*D*		~~0.90~~ 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*D*~~ 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*D*~~ 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

Modify the 1st Paragraph as follows:

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~~3~~, and, when appropriate, by considering reduction due to effects of uplift.

C10.8.3.7.2

Modify the 1st Paragraph as follows:

The side resistance ~~factors~~ for uplift ~~are~~ is lower than that ~~those~~ for axial compression. One reason for this is that drilled shafts in tension unload ~~the~~ soils, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft. ~~Empirical justification for uplift resistance factors is provided in Article C10.5.5.2.3, and in Allen (2005).~~

Revise title as follows:

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

Revise as follows:

Center-to-center ~~pile~~ spacing of micropiles should 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 in.

This page is intentionally left blank.

10.9.3.5.4—Micropile Load Test

Delete the entire article and replace with the following:

Section 49-5 of the *Standard Specifications* and the project special provisions shall supersede Article 10.9.3.5.4.

C10.9.3.5.4

Delete the entire commentary and replace with the following:

Section 49-5 of the *Standard Specifications* and the project special provisions shall supersede Article C10.9.3.5.4.

This page is intentionally left blank.

11.3 - Notation

Add the following definitions:

D_{min} = distance between the back of MSE facing elements and any concrete footing element (11.10.11)

H_{max} = distance between superstructure soffit and finished grade in front of the MSE facing (11.10.11)

This page intentionally left blank

11.5.1—General

Revise 2nd Paragraph of Article 11.5.1 as follows:

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.

Revise the 3rd Paragraph as follows:

Earth retaining structures shall be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potential deleterious environmental factors on each of the material components comprising the structure. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining wall applications defined as temporary shall be considered to have a service life of ~~36 months~~ 5 years or less.

This page intentionally left blank

11.5.6—Load Combinations and Load Factors

Revise Article 11.5.6 as follows:

~~Abutments, p~~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.

This page intentionally left blank

Modify Table

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		~~0.75~~ 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 A615 bars) • High strength steel (e.g., ASTM A722 bars) • High strength steel strands (e.g. ASTM A416)	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	 ~~0.75~~ 0.90 ~~0.65~~ 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

(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) Apple to maximum proof test load for the anchor. For mild steel apply resistance factor to Fy.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 hole, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

(5) Applies to grid reinforcement 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.

This page intentionally left blank

11.6.1.5.2—Wingwalls

Modify as follows:

Reinforcement 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.

11.6.1.6—Expansion and Contraction Joints

Modify as follows:

Weakened plane ~~Contraction~~ joints shall be provided at intervals not exceeding 24.0 ~~30.0~~ ft and expansion joints at intervals not exceeding 96.0 ~~90.0~~ ft for conventional retaining walls and abutments. All joints shall be filled with approved filling material to ensure the function of the joint. Joints in abutments shall be located approximately midway between the longitudinal members bearing on the abutments.

This page intentionally left blank

Revise Title:

11.6.5—Seismic Design for ~~Abutments and~~ Conventional Retaining Walls

C11.6.5

Add Article C11.6.5 as follows:

Abutments founded in competent soil 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.

11.6.5.1—General

Revise: 1st sentence of the 1st paragraph as follows:

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

Delete the 3rd paragraph of Article 11.6.5.1:

~~For bridge abutments, the abutment seismic design should be conducted in accordance with Articles 5.2 and 6.7 of AASHTO's Guide Specifications for LRFD Seismic Bridge Design but with the following exceptions:~~

- ~~K_h should be determined as specified in Article 11.6.5.2 and~~
- ~~Lateral earth pressure should be estimated in accordance with Article 11.6.5.3~~

Revise Title:

11.6.5.4—Calculation of Seismic Earth Pressure for Nonyielding ~~Abutments and~~ Walls

Revise 1st sentence of the 1st paragraph as follows:

For ~~abutments walls and other~~ 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.

This page intentionally left blank

11.10.6.4.2a—Steel Reinforcements

Add the following before Paragraph 4:

When soil backfill conforms to the following criteria:

• pH = 5 to 10

• Resistivity ≥ 2000 ohm-cm

• Chlorides ≤ 250 ppm

• Sulfates ≤ 500 ppm

• Organic Content ≤ 1 percent

Sacrificial thicknesses shall be computed for each exposed surface as follows:

• Loss of galvanizing takes 10 years
• Loss of carbon steel = 1.1 mil./yr. after zinc depletion

C11.10.6.4.2a

Add a new paragraph to Article after Paragraph 4:

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.

This page intentionally left blank

11.10.11— MSE Abutments

Modify the following text in the 6th Paragraph:

~~The minimum distance from the centerline of the bearing on the abutment to the outer edge of the facing shall be 3.5 ft.~~ The minimum soil cover over the footing shall be 2.0 ft. The minimum thickness of compacted backfill between the concrete footing elements and the soil reinforcement shall be 4 inches. The minimum distance, D_{min}, between the back ~~face of the panel and the~~ of the MSE facing elements and any element of the concrete footing shall be ~~6.0 in~~ as follows

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

Where H_{max} is the clear distance between the superstructure soffit and the finished grade in front of the MSE facing. The maximum clearance, H_{max}, shall be 30 ft.

Modify the following text in the 9th Paragraph:

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 ~~1.5~~ 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.

This page intentionally left blank

12.6.6—Soil Envelope

12.6.6.1—Trench Installations

Revise the 1st Paragraph as follows:

The minimum trench width shall provide a 24-in. minimum side wall clearance ~~sufficient space~~ 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

Revise the 1st and 2nd Paragraphs as follows:

~~As a guide, the minimum trench width should not be less than the greater of the pipe diameter plus 16.0 in.or the pipe diameter times 1.5 plus 12.0 in.~~ 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.

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 in. in diameter or span, or 12 in. for pipes over 42 in. in diameter or span.

Revise Table C12.6.6.2-1 as follows:

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

Diameter, *S* (in.)	Minimum Envelope Width (ft)
<24	*S*/12
24-~~144~~ 108	2.0
>~~144~~ 108	5.0

Revise Table 12.6.6.3-1 as follows:

Table 12.6.6.3-1—Minimum Cover

Type	Condition	Minimum Cover*
Corrugated Metal Pipe		$S/8 \geq$ ~~12.0 in.~~ 24.0 in.
Spiral Rib Metal Pipe	Steel Conduit	$S/4 \geq$ ~~12.0 in~~. 24.0 in.
	Aluminum Conduit where $S \leq 48.0$ in.	$S/2 \geq$ ~~12.0 in.~~ 24.0 in.
	Aluminum Conduit where $S > 48.0$ in.	$S/2.75 \geq 24.0$ in.
Structural Plate Pipe Structures	—	$S/8 \geq$ ~~12.0 in.~~ 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 Structure Plate Structures	—	See Article 12.8.9.4
Thermoplastic Pipe	Under unpaved areas	$ID/8 \geq$ ~~12.0 in.~~ 24.0 in.
	Under paved roads	$ID/2 \geq 24.0$ in.
* Minimum cover taken from top of rigid pavement or bottom of flexible pavement		
Type	Condition	Minimum Cover
Reinforced Concrete Pipe	Under unpaved areas or top of flexible pavement	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ ~~12.0 in.~~ 24.0 in.
Type	Condition	Minimum Cover
Reinforced Concrete Pipe	Under bottom of rigid pavement	~~9.0 in.~~ 12.0 in.

This page is intentionally left blank.

Revise Table 12.10.2.1-1 as follows:

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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ 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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ 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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ in. minimum, not less than 6.0 in.	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL
~~Type 4~~	~~For soil foundation, no bedding required. For rock foundation, use Bc/2 ft minimum, not less than 6.0 in.~~	~~No compaction required, except if CL, use 85% CL~~	~~No compaction required, except if CL, use 85% CL~~

This page is intentionally left blank.

Revise Table 12.10.2.1-2 as follows:

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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ 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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ in. minimum, not less than 6.0 in.	90% SW or 95% ML	85% SW, 90% ML, or 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 ~~ft~~ in. minimum, not less than 3.0 in. For rock foundation, use B_c ~~ft~~ in. minimum, not less than 6.0 in.	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL, or natural soils of equal firmness
~~Type 4~~	~~For soil foundation, no bedding required. For rock foundation, use B_c /2 ft minimum, not less than 6.0 in.~~	~~No compaction required, except if CL, use 85% CL~~	~~85% SW, 90% ML, 95% CL, or natural soils of equal firmness~~

This page is intentionally left blank.

12.10.2.1—Standard Installations

Revise Table 12.10.2.1-3 as follows:

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

	Installation Type			
	1	2	3	~~4~~
VAF	1.35	1.40	1.40	~~1.45~~
HAF	0.45	0.40	0.37	~~0.30~~
A1	0.62	0.85	1.05	~~1.45~~
A2	0.73	0.55	0.35	~~0.00~~
A3	1.35	1.40	1.40	~~1.45~~
A4	0.19	0.15	0.10	~~0.00~~
A5	0.08	0.08	0.10	~~0.11~~
A6	0.18	0.17	0.17	~~0.19~~
a	1.40	1.45	1.45	~~1.45~~
b	0.40	0.40	0.36	~~0.30~~
c	0.18	0.19	0.20	~~0.25~~
e	0.08	0.10	0.12	~~0.00~~
f	0.05	0.05	0.05	—
u	0.80	0.82	0.85	~~0.90~~
v	0.80	0.70	0.60	—

Add an additional paragraph and three figures after the last paragraph as follows:

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 lbs/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 Services. 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 12.10.2.1-2(a), or the Paris/Manual Uniform Pressure Distribution - see Figure 12.10.2.1-2(b). For bedding angles and lateral pressures used with the latter distributions see Figure 12.10.2.1-3 and Figure 12.10.2.1-4. 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.

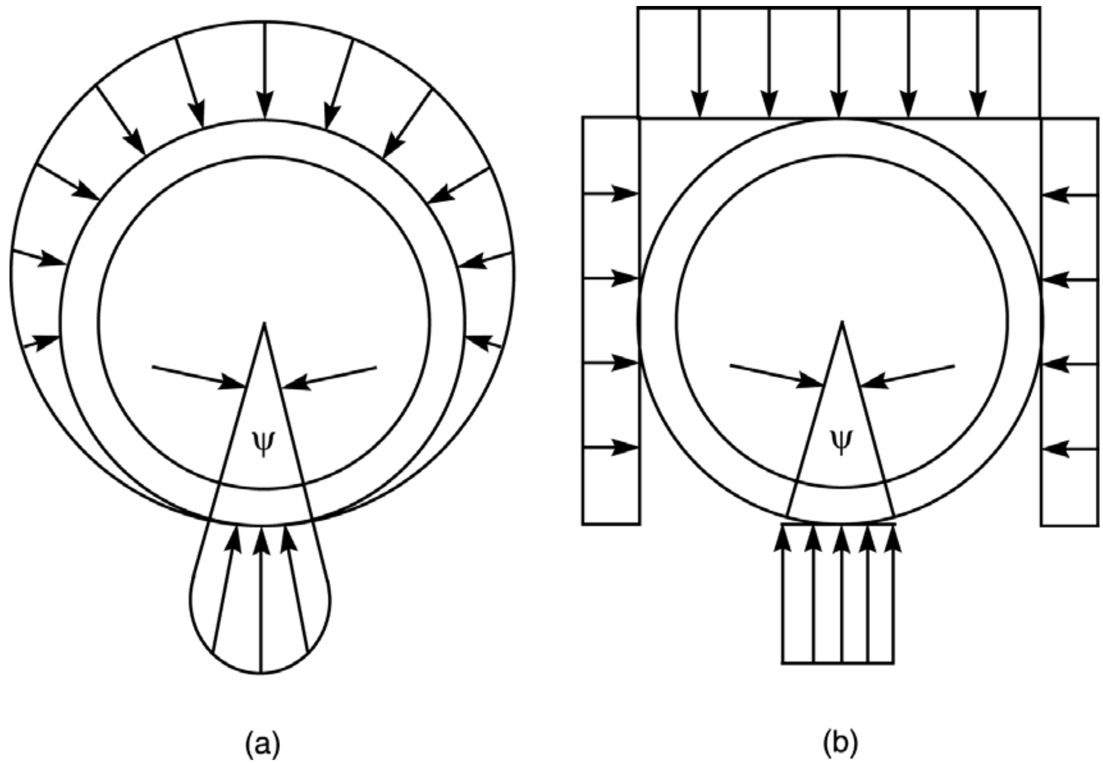


Figure 12.10.2.1-2(a)—Olander/Modified Olander Radial Pressure Distribution Diagram
Figure 12.10.2.1-2(b)—Paris/Manual Uniform Pressure Distribution Diagram

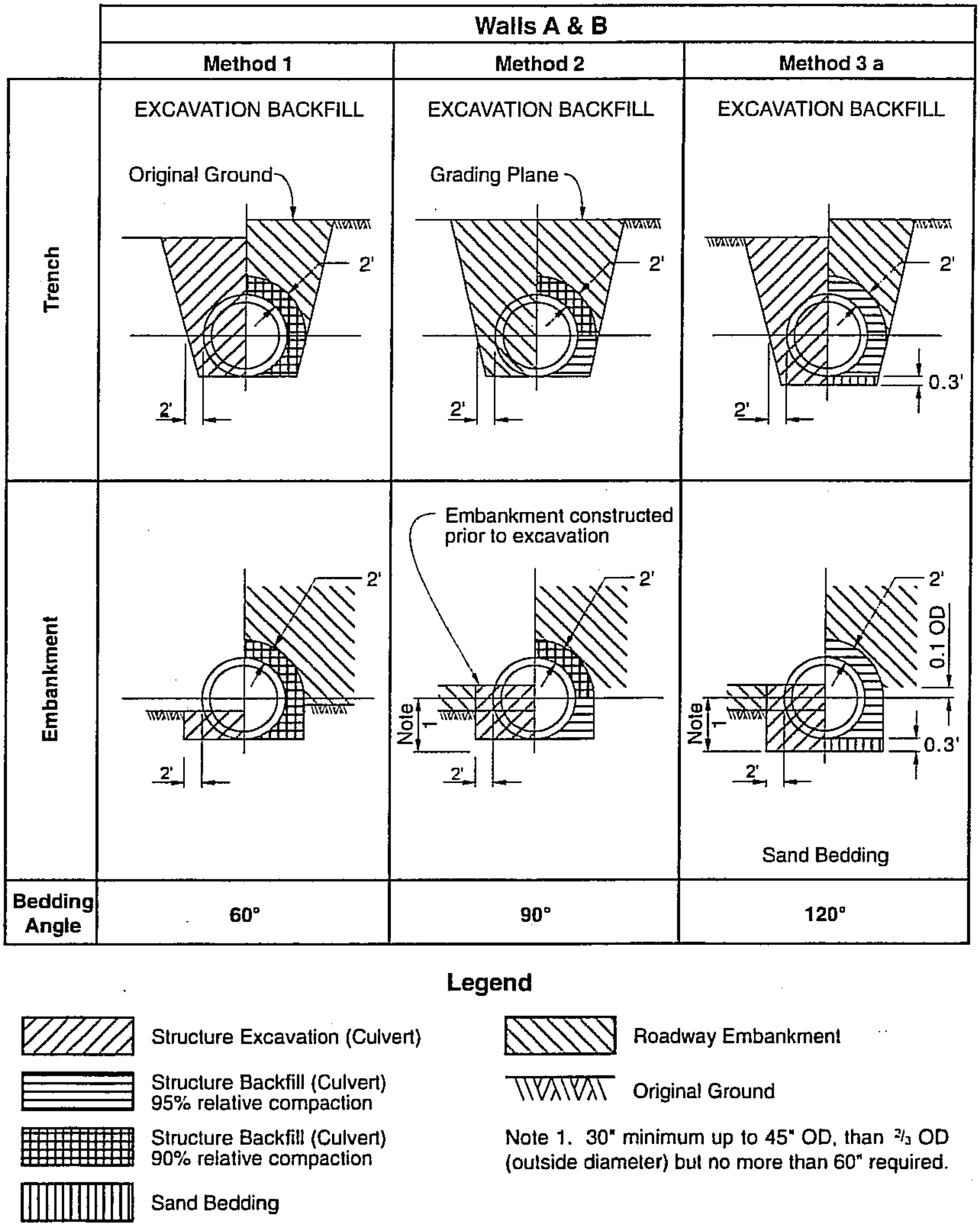
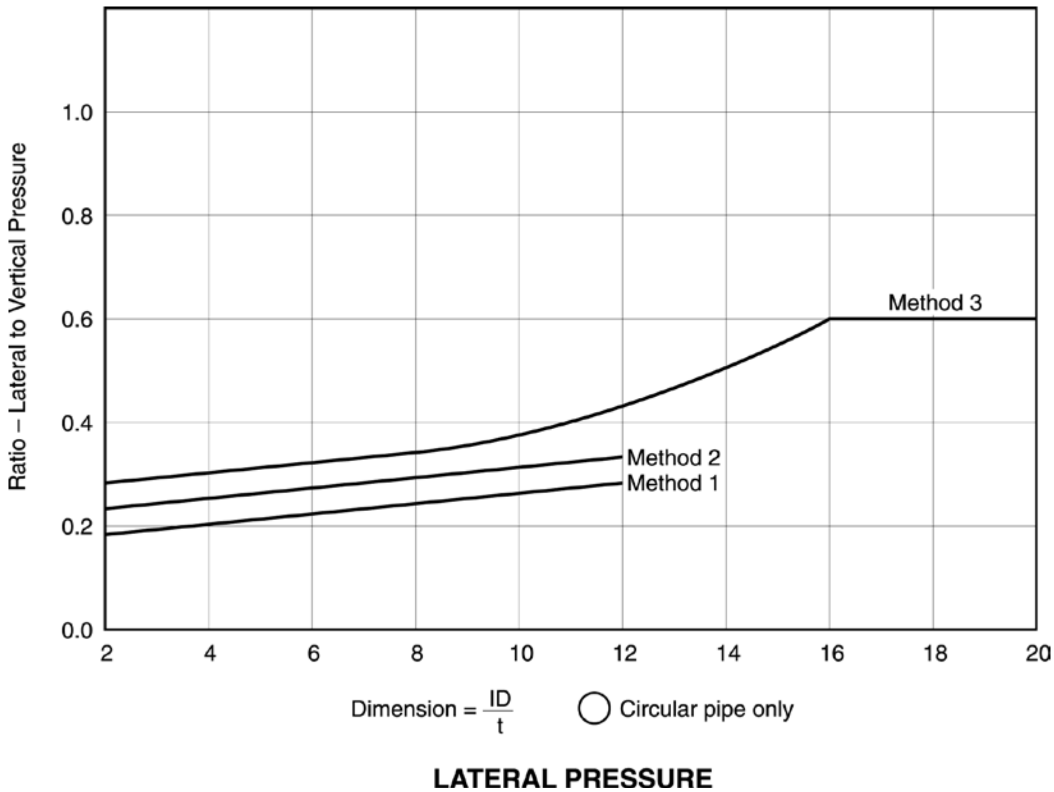


Figure 12.10.2.1-3—Trench and Embankment Backfill Bedding Angles



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

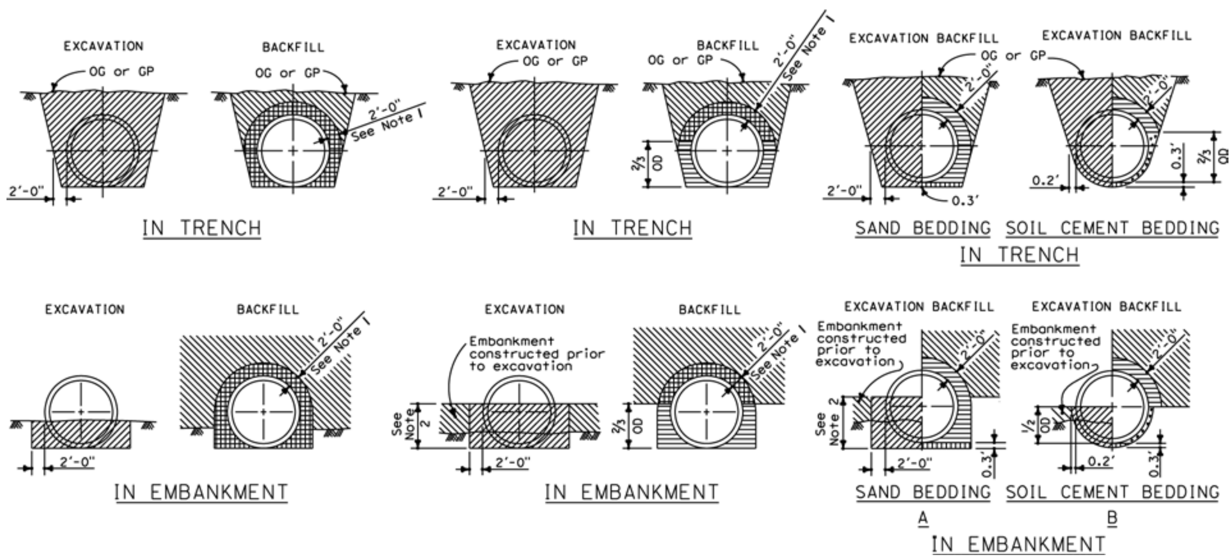
Figure 12.10.2.1-4—Non-Standard Installation Lateral Pressures Distribution

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. Above information is based on Caltrans research *(Transportation Record 878)*, and Caltrans *Standard Plans 2010*, A62D.



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

Note:

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

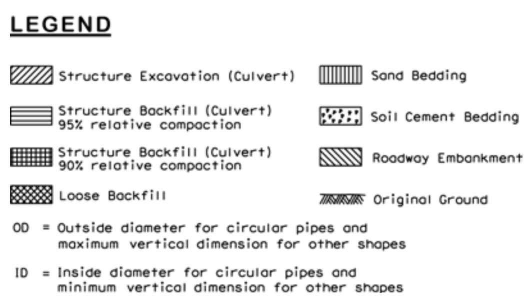


Figure 12.10.4.3.1-1—Non-Standard Installation Excavation Backfill

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

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

This page is intentionally left blank.

12.10.4.3.2a—EarthLoad Bedding Factor for Circular Pipe

Revise Table 12.10.4.3.2a-1 as follow:

Table 12.10.4.3.2a-1—Bedding Factors for Circular Pipe

Pipe Diameter, in.	Standard Installations			
	Type 1	Type 2	Type 3	~~Type 4~~
12	4.4	3.2	2.5	~~1.7~~
24	4.2	3.0	2.4	~~1.7~~
36	4.0	2.9	2.3	~~1.7~~
72	3.8	2.8	2.2	~~1.7~~
144	3.6	2.8	2.2	~~1.7~~

This page is intentionally left blank.

12.15—REFERENCES

Add the following references:

Alfred E. Bacher, Albert N. Banke, and Daniel E. Kirkland. 1963. "Reinforced Concrete Pipe Culverts: Design Summary and Implementation." *Transportation Record 878.* Committee on Culverts and Hydraulic Structures, California Department of Transportation, Sacramento, CA.

Caltrans, *Standard Plans 2010*, California Department of Transportation, Sacramento, CA.

Caltrans, *Standard Specifications 2010*, California Department of Transportation, Sacramento, CA.

Caltrans, *Bridge Design Specifications, LFD Version*, April 2000 Section 17 – Soil Reinforced Concrete Structure Interaction Systems, California Department of Transportation, Sacramento, CA.

Caltrans, *CA Test* 216 Method of Test for Relative Compaction of Untreated and Treated Soils and Aggregates, California Department of Transportation, Sacramento, CA.

Caltrans, *CA Test* 231 Method of Test for Relative Compaction of Untreated and Treated Soils and Aggregates Using Nuclear Gage, California Department of Transportation, Sacramento, CA.

This page is intentionally left blank.

13.9.2–Geometry

Revise as follows:

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 shall not be less than 48.0 in. measured from the top of the riding surface.

The height of the upper and lower zones of a bicycle railing shall be at least 27.0 in. The upper and lower zones 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

Add new Paragraphs 2 and 3:

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.

This page intentionally left blank

A13.4.2 – Decks Supporting Concrete Parapet Railings

Revise as shown below:

For Design Case 1, the deck overhang shall ~~may~~ be designed to resist ~~provide a flexural resistance, M_s in kip-ft./ft. which, acting coincident with~~ the combined effects of tensile force T in kip/ft, and moment M_{ct} as specified herein, ~~exceeds M_c of the parapet at its base. The axial tensile force T, may be taken as:~~

$$\cancel{T = \frac{R_w}{L_c + 2H}}$$

$$T = 1.2\left(\frac{F_t}{L_c}\right) \quad \text{(A13.4.2-1)}$$

$$M_{cr} = 1.2\left[\frac{F_t\,H}{L_c}\right] \quad \text{(A13.4.2-2)}$$

where:

~~R_w = parapet resistance specified in Article 13.3.1 (kips)~~

L_c = critical length of yield line failure pattern (ft). In the absence of more accurate calculations, L_c, may be taken as 10 ft for solid concrete parapets ; this value of L_c is valid for design forces TL - 1 through TL - 4 shown in Table A13.2-1. At the location of expansion joints, the value of L_c shall be half that specified above.

H = height of wall (ft)

T = tensile force per unit of deck length (kip/ft)

M_{cr} = moment in the deck overhang due to F_t (kip/ft-ft)

CA13.4.2

Delete the 1st and 2nd Paragraphs and replace with the following:

In the design of barrier rails, it is recognized that the crash testing program is oriented towards survival, not necessarily the identification of the ultimate strength of the railing system. This typically produces a railing system that is significantly overdesigned, and in turn would lead to an over-design of the deck overhang that may not be practical.

Therefore, the design of a deck overhang for Design Case 1 is based on F_t - the transverse force on the barrier rail corresponding to the Test Level as shown in Table A13.2-1, not on the capacity of the barrier rail. To account for uncertainties in the load and mechanisms of failure, and to provide an adequate safety margin, the actual design tensile force acting on the deck overhang and the corresponding design moment obtained through statics are increased by 20%.

All deck overhangs should be designed for TL-4 Barrier Rail loading.

At an expansion joint, and at the beginning and end of a bridge, the value of L_c will be half that at intermediate locations. This will cause an increase in force effects in the overhang region. Consequently, the top reinforcing bars in the overhang should be designed to accommodate this increased force effect in this region.

This page intentionally left blank

14.7.5.3.4—Stability of Elastomeric Bearings
Replace Article 14.7.5.3.4:

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

~~Bearings satisfying Eq. 14.7.5.3.4-1 shall be considered stable, and no further investigation of stability is required.~~

$$\cancel{2A \le B} \qquad \text{~~(14.7.5.3.4-1)~~}$$

~~in which:~~

$$\cancel{A = \frac{1.92\frac{h_{rt}}{L}}{\sqrt{1+\frac{2.0L}{W}}}} \qquad \text{~~(14.7.5.3.4-2)~~}$$

$$\cancel{B = \frac{2.67}{(S_i + 2.0)(1+\frac{L}{4.0W})}} \qquad \text{~~(14.7.5.3.4-3)~~}$$

~~where:~~

~~G = shear modulus of the elastomer (ksi)~~
~~h_{rt} = total elastomer thickness (in.)~~
~~L = plan dimension of the bearing perpendicular to the axis of rotation under consideration (generally parallel to the global longitudinal bridge axis) (in.)~~
~~S_i = shape factor of the i^{th} internal layer of an elastomeric bearing.~~
~~W = plan dimension of the bearing parallel to the axis of rotation under consideration (generally parallel to the global transverse bridge axis) (in.)~~

~~For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W in Eqs. 14.7.5.3.4-2 and 14.7.5.3.4-3.~~

~~For circular bearings, stability may be investigated by using the equations for a square bearing with W=L=0.8L).~~

~~For rectangular bearings not satisfying Eq. 14.7.5.3.4-1, the stress due to the total load shall satisfy Eq. 14.7.5.3.4-4 or 14.7.5.3.4-5.~~

- ~~If the bridge deck is free to translate horizontally:~~

$$\cancel{\sigma_s \le \frac{GS_i}{2A-B}} \qquad \text{~~(14.7.5.3.4-4)~~}$$

C14.7.5.3.4
Replace Article C14.7.5.3.4:

~~The average compressive stress is limited to half the predicted buckling stress. The latter is calculated using the buckling theory developed by Gent, modified to account for changes in geometry during compression, and calibrated against experimental results (Gent, 1964; Stanton at al., 1990). This provision will permit taller bearings and reduced shear forces compared to those permitted under previous editions of the AASHTO Standard Specifications.~~

~~Eq. 14.7.5.3.4-4 corresponds to buckling in a sideway mode and is relevant for bridges in which the deck is not rigidly fixed against horizontal translation at any point. This may be the case in many bridges for transverse perpendicular to the longitudinal axis. If one point on the bridge is fixed against horizontal movement, the sideway buckling mode is not possible, and Eq. 14.7.5.3.4-5 should be used. This freedom to move horizontally should be distinguished from the question of whether the bearing is subject to shear deformations relevant to Articles 14.7.5.3.2 and 14.7.5.3.3. In a bridge that is fixed at one end, the bearings at the other end will be subjected to impose shear deformation but will not be free to translate in the sense relevant to buckling due to the restraint at the opposite end of the bridge.~~

~~A negative or infinite limit from Eq. 14.7.5.3.4. 5 indicates that the bearing is stable and is not dependent on σ_s.~~

~~If the value $A-b \le 0$, the bearing is stable and is not dependent on σ_s.~~

<u>Equation (14.7.5.3.4-3) presumes that the bridge is not rigidly fixed against horizontal translation in the longitudinal direction. Buckling in the transverse bridge direction is not considered because either the direction is restrained, or if not, longitudinal buckling dominates due to the placement of bearings with the long dimension perpendicular to the bridge longitudinal axis. In any case, the designer should check buckling for the governing scenario.</u>

- ~~If the bridge deck is fixed against horizontal translation:~~

$$\cancel{\sigma_s \le \frac{GS_i}{A-B}} \qquad \text{~~(14.7.5.3.4-5)~~}$$

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

$$P'_{cr_s} = 0.680 \frac{GBL^2(L - \Delta_S)}{(1 + L/B)tT_r} \qquad (14.7.5.3.4\text{-}3)$$

A bearing design may be considered acceptable for buckling if

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

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 HL-93 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.)

This page intentionally left blank

14.10 –REFERENCES

Add to References:

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

This page intentionally left blank