3.3.2—Load and Load Designation

Add the following notations:

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

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3.4.1—Load Factors and Load Combinations

Replace the following notation in the 1st paragraph:

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

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

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

C3.4.1

Replace the 2nd paragraph with the following:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Replace Table 3.4.1-1 with the following:

Table 3.4.1-1—Load Combinations and Load Factors

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

Add Article 3.4.5 as follows:

3.4.5—Load Factors for Abutments

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

3.4.5.1—Service, Strength, and Construction Load Combinations

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

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

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

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

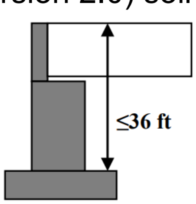
Type of Load and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC_{Sub}: Dead Load of Structure Components and Nonstructural Attachments of Substructure		1.25	0.90
DC_{Sup}: Dead Load of Structure Components and Nonstructural Attachments of Superstructure		1.25	0.90
DD: Downdrag	Pile,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

3.4.5.2—Extreme Event-I (Seismic) Load Combination

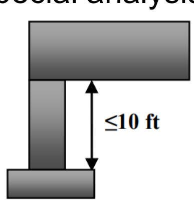
If an abutment in Type S1 (as defined in Article 6.1.2 of SDC version 2.0) soil meets the following height limitations, seismic forces shall be considered **only** in global stability analysis of the abutment:

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

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



Non-Integral Type Abutment (with/without piles)



Integral Type Abutment (with/without piles)

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

Add the following after the 2nd paragraph:

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

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

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3.6.1.2.6a—General

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

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

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

Replace Table 3.6.1.2.6a-1 with the following:

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

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

3.6.1.2.6b—Traffic Parallel to the Culvert Span

Replace the equation 3.6.1.2.6b-1 with the following:

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

Replace equation 3.6.1.2.6b-6 with the following:

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

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

3.6.1.3.1—General

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

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

C3.6.1.3.1

Replace the 3rd paragraph with the following:

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

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

C3.6.1.3.3

Add a new 5th paragraph as follows:

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

3.6.1.4—Fatigue Load

3.6.1.4.1—Magnitude and Configuration

Replace the 1st paragraph with the following:

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

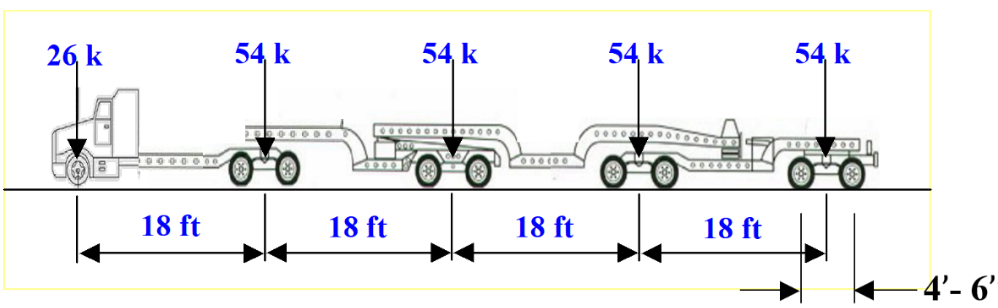
Add the following after the 2nd paragraph:

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

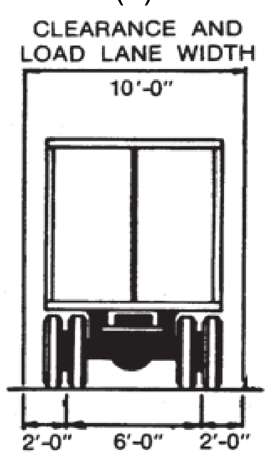
C3.6.1.4.1

Add the following paragraph:

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



(a)



(b)

Figure 3.6.1.4.1-2 — Permit Truck, P9

3.6.1.4.2—Frequency

Add the following as the last 2 paragraphs:

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

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

C 3.6.1.4.2

Add the following as the last paragraph:

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

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3.6.1.6—Pedestrian Loads

Replace the article with the following:

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

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

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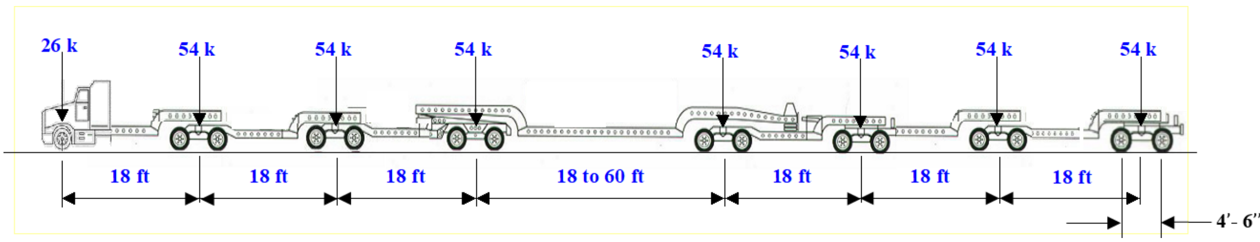
Add Article 3.6.1.8 as follows:

3.6.1.8—Permit Vehicle: LL_{permit}

3.6.1.8.1—General

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



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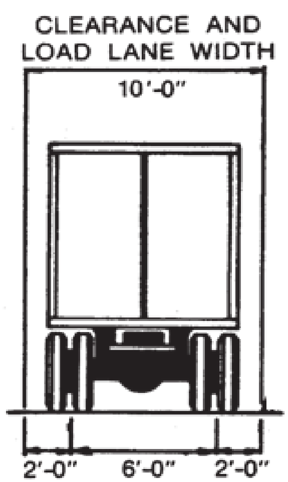

(b)

Figure 3.6.1.8.1—1 Permit Truck, P15

3.6.1.8.2—Application

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

Add the commentary as follows:

C3.6.1.8

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

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

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

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

Centrifugal force shall be applied as specified in Article 3.6.3.

3.6.2—Dynamic Load Allowance: *IM*

3.6.2.1—General

Replace the 1st paragraph with the following:

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

Replace Table 3.6.2.1-1 with the following:

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

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

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

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

C3.6.2.1

Replace the 4th paragraph with the following:

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

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Replace the 6th paragraph with the following:

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

Replace the 7th paragraph with the following:

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

3.6.3—Centrifugal Forces: *CE*

Replace the 1st paragraph with the following:

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

(no change to equation)

Replace the 2nd paragraph with the following:

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

Replace the 4th paragraph with the following:

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

C3.6.3

Replace the 4th paragraph with the following:

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

3.6.4—Braking Force: *BR*

Replace the 2nd paragraph with the following:

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

C3.6.4

Replace 1st paragraph with the following:

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

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

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

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3.6.5—Vehicular Collision Force: *CT*

3.6.5.1—Protection of Structures

Replace the 2nd paragraph with the following:

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

C3.6.5.1

Add a new paragraph to the beginning of the commentary:

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

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3.7.5—Change in Foundations Due to Limit State for Scour

Replace the article with the following:

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

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

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

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

C3.7.5

Replace the 2nd paragraph with the following:

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

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3.8.1.3—Wind Load on Live Load: *WL*

Replace the 1st paragraph with the following:

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

C3.8.1.3

Add a new 3rd paragraph as follows:

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

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3.10—EARTHQUAKE EFFECTS: EQ

Add a new paragraph as follows:

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

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3.12.2—Uniform Temperature

Replace the article with the following:

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

3.12.2.1—Temperature Range for Procedure A

Replace the 1st paragraph with the following:

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

Replace the 2nd paragraph with the following:

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

Add a 3rd paragraph as follows:

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

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

where:

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

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

3.12.2.2—Temperature Range for Procedure B

Delete the entire article and commentary.

3.12.2.3—Design Thermal Movements

Replace the article with the following:

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

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

where:

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

Add a new commentary as follows:

C3.12.2.3

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

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