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.

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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	$\cancel{\gamma_p}$ 1.00	γEQ	—	1.00	—	—	1.00	—	—	—	1.00	---	—	—	—
EXTREME EVENT II	$\cancel{\gamma_p}$ 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.

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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.

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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.

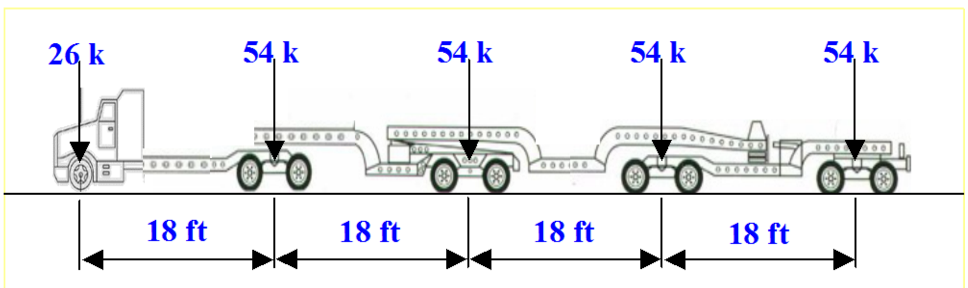

Figure 3.6.1.4.1-2 Fatigue Permit Truck

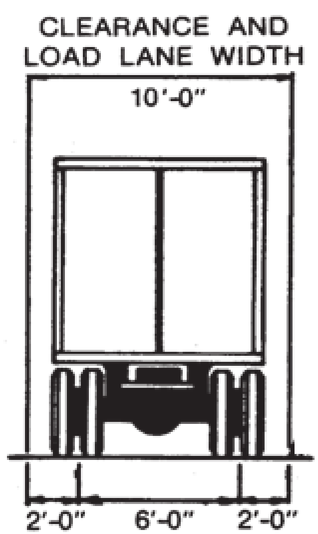

Figure 3.6.1.4.1-3 Fatigue Permit Truck

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.

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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.

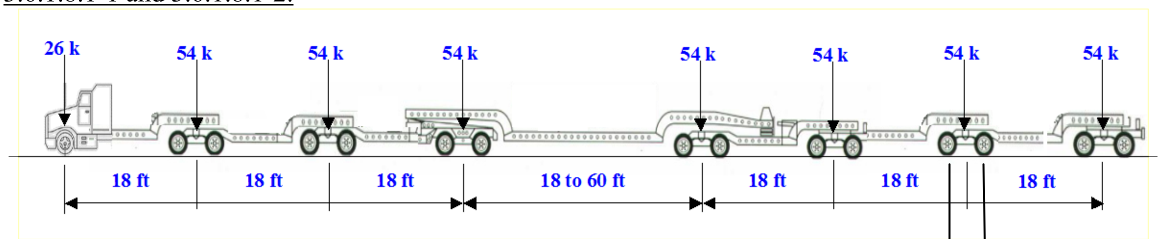

Figure 3.6.1.8.1-1—California P15 truck

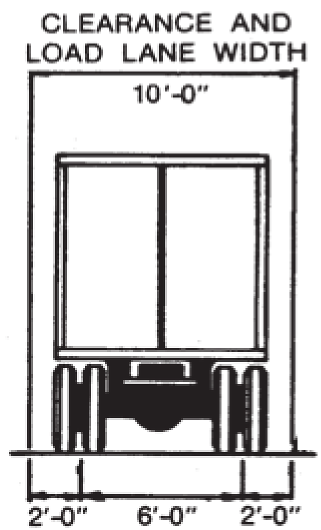

Figure 3.6.1.8.1-2 —California P15 truck gage

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%
• Strength II Limit State	25%
• 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. 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.

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.

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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.

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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.

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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.

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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:

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

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.

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