

3.2 DEFINITIONS

Revise the following definition:

Permanent Loads—Loads and forces that are, or are assumed to be either constant upon completion of construction or varying only over a long time interval.

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3.2 DEFINITIONS

Add the following definition:

Transient Loads—Loads and forces that can vary over a short time interval relative to the lifetime of the structure.

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3.3.1 General

Add the following notation:

$T_{BaseConstr}$ = base construction temperature

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3.3.2 Load and Load Designation

Revise the following definitions:

- Permanent Loads

CR = force effect due to creep

DD = downdrag force

EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction

PS = secondary forces from post-tensioning

SH = force effect due to shrinkage

- Transient Loads

~~CR~~ = creep

EQ = earthquake load

FR = friction force

SE = force effect due to settlement

~~SH~~ = shrinkage

TG = force effect due to temperature gradient

TU = force effect due to uniform temperature

3.4.1 Load Factors and Load Combinations

Revise as follows:

where:

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

Revise the 2nd Bullet 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) Load combination applies to superstructure design with load distribution factor tables in Articles 4.6.2.2, only.
 - b) Load combination used for 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 ~~are~~ is to be used. Live loads shall be placed in a maximum of two separate lanes chosen to create the most severe conditions.

C3.4.1

Revise the 3rd Paragraph 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 4.6.2.2.4 regarding other traffic on the bridge simultaneously. The vehicular braking force shall not be included in this load combination.~~

- a) Multiple presence is already considered in the load distribution factor tables in Articles 4.6.2.2.
- b) Multiple presence factors from Article 3.6.1.1.2 apply if the lever rule is used and in the case of substructure components.

C3.4.1

Revise the 7th Paragraph as follows:

Although this limit state includes water loads, *WA*, the effects due to *WA* are considerably less significant than the effects on the structure stability due to degradation and contraction scour. Therefore, unless specific site conditions dictate otherwise, local pier scour ~~and contraction scour~~ depths should not be included in the structural or geotechnical design. However the effects due to degradation and contraction scour of the channel should be considered. Live load coincident with an earthquake is discussed elsewhere in this article.

Revise the 9th Paragraph 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 is expected to 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 scour and contraction scour only, should be considered for both structural and geotechnical design.

3.4.1 Load Factors and Load Combinations

Revise the 12th Bullet as follows:

- FATIGUE I — Fatigue and fracture load combination relating to finite fatigue life and infinite fatigue life due to repetitive gravitational vehicular HL-93 truck live load and dynamic response under a single design truck having the axle spacing specified in Article 3.6.1.4.1. The load factors of 0.875 and 1.75 shall be used for finite fatigue life and infinite fatigue life, respectively.

C3.4.1

Revise the 17th Paragraph as follows:

The load factor applied to a single design truck, reflects a load level found to be representative of the truck population with respect to a large number of return cycles of stresses and to their cumulative effects in steel elements, components, and connections.

Add the following after the 17th 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. 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.

A comprehensive comparison study of fatigue load moments for steel girder bridges using the AASHTO LRFD Bridge Design Specifications (3rd Edition, 2004) compared to 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 0.875 together with a revised Fatigue Resistance Equation (6.6.1.2.5-1a), and the load factor of 1.75 together with a revised Fatigue Resistance Equation (6.6.1.2.5-1b) should be used for ~~infinite~~ finite fatigue life and infinite fatigue life in Fatigue I Limit State, respectively, for steel design. Those factors are based on the assumption that the maximum stress range is twice the live load stress range 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-kips axles and derived by calibrating the LRFD fatigue design procedure to Caltrans past LFD design procedure. Figure C1 shows comparisons of fatigue resistance vs. number of cycles for a steel detail.

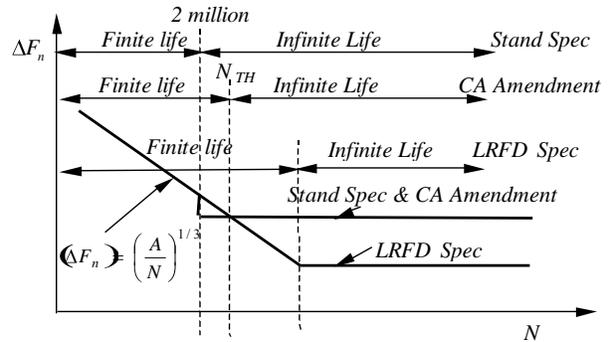


Figure C3.4.1-1 Fatigue Resistance vs. Number of Cycles.

3.4.1 Load Factors and Load Combinations

Add the following after the 12th Bullet:

- FATIGUE II — Fatigue and fracture load combination relating to finite fatigue life due to repetitive Fatigue Permit truck live load and dynamic response under a single design truck having the axle spacing specified in Article 3.6.1.4.1.

Revise the 6th Paragraph as follows:

The larger of the two values provided for load factors of *TU*, *CR*, and *SH* shall be used for deformations and the smaller values for all other effects. For simplified analysis of substructures in the strength limit state, a value of 0.50 for γ_{TU} may be used when calculating force effects, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. When a refined analysis is completed for substructures in the strength limit state, a value of 1.0 for γ_{TU} shall be used in conjunction with a partially cracked moment of inertia determined by analysis. For concrete substructures in the strength limit state, the value of 0.50 for γ_{PS} , γ_{CR} , and γ_{SH} may similarly be used when calculating force effects in non-segmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. For steel substructures, a value of 1.0 for γ_{TU} , γ_{PS} , γ_{CR} , and γ_{SH} shall be used.

C3.4.1

Add the following before the 18th Paragraph:

The load factor of 1.0 applied to a single design truck, reflects a load level found to be representative of the permit truck population with respect to a large number of return cycles of stresses and to their cumulative effects in elements, components, and connections.

Add the following after the 19th paragraph:

Load factors for earth loads have traditionally been set at about 2.0 for flexible culverts and 1.3 for rigid culverts. Since the uncertainty of earth load magnitude for flexible and rigid culverts should be similar, the earth load factor is set to 1.3; however, to preserve the overall safety at the same levels as historical specifications, an earth-load-installation factor is introduced later in these specifications. This factor may be adjusted based on field control of construction practices.

3.4.1 Load Factors and Load Combinations

Revise Eq. 3.4.1-2 as follows:

$$\underline{DC + DW + EH + EV + ES + WA + CR + SH + TG + EL} + \underline{PS}$$

(3.4.1-2)

C3.4.1

Add the following after the 25th Paragraph:

PS, CR, SH, TU and TG are superimposed deformations as defined in Article 3.12. Load factors for TU and TG are as shown in Table 1. Load factors for PS, CR, and SH, are as shown in Table 3. For prestressed members in typical bridge types, secondary prestressing, creep and shrinkage are generally designed for in the service limit state. In segmental structures, CR and SH are factored by γ_p for DC because some analytic methods for time-dependent effects in segmental bridges are nonlinear. The calculation of displacements for TU utilizes a factor greater than 1.0 to avoid undersizing joints, expansion devices, and bearings

Table 3.4.1-1 Load Combination and Load Factors.

Replace 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 _{HL93} IM CE BR PL LS	LL _{Permit} IM CE	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
											EQ	IC	CT	CV
STRENGTH I	γ_p	1.75	—	1.00	—	—	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	—	<u>1.35</u>	1.00	—	—	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	—	1.00	1.40	—	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	—	1.00	—	—	1.0	0.50/ 1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	—	1.00	0.40	1.0	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	<u>1.00</u>	γ_{EQ}	—	1.00	—	—	1.0	—	—	—	1.00	—	—	—
EXTREME EVENT II	<u>1.00</u>	0.5	—	1.00	—	—	1.0	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	—	1.00	0.30	1.0	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	—	1.00	—	—	1.0	1.00/ 1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	—	1.00	—	—	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	—	1.00	0.70	—	1.0	1.00/ 1.20	—	1.0	—	—	—	—
FATIGUE I - LL _{HL93} , IM & CE ONLY	—	<u>0.875/ 1.75</u>	—	—	—	—	—	—	—	—	—	—	—	—
FATIGUE II - LL _{Permit} & IM & CE	—	—	<u>1.00</u>	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p .

Replace with the following:

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p .

Add the following bullet under ~~EV~~, Vertical Earth Pressure”:

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
<i>DC</i> : Component and Attachments		1.25	0.90
<i>DC</i> : Strength IV, only		1.50	0.90
<i>DD</i> : Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled Shafts, O’Neill and Reese (1999) Method	1.25	0.35
<i>DW</i> : Wearing Surfaces and Utilities		1.50	0.65
<i>EH</i> : Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• <i>AEP</i> for Anchored Walls		1.35	N/A
<i>EL</i> : Locked-in Construction Stresses		1.00	1.00
<i>EV</i> : Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
• <u>Thermoplastic Culverts</u>		<u>1.30</u>	<u>0.90</u>
<i>ES</i> : Earth Surcharge		1.50	0.75

Table 3.4.1-3 Load Factors for Permanent Loads due to Superimposed Deformations, γ_p .

Insert the following after Table 3.4.1-2:

Table 3.4.1-3 Load Factors for Permanent Loads due to Superimposed Deformations, γ_p .

Bridge Component	<i>PS</i>	<i>CR,SH</i>
<u>Superstructures–Segmental</u>	<u>1.0</u>	<u>See γ_p for <i>DC</i>, Table 2</u>
<u>Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)</u>		
<u>Concrete Superstructures–non-segmental</u>	<u>1.0</u>	<u>1.0</u>
<u>Substructures supporting non-segmental Superstructures</u>		
• using I_g	<u>0.5</u>	<u>0.5</u>
• using $I_{effective}$	<u>1.0</u>	<u>1.0</u>
<u>Steel Substructures</u>	<u>1.0</u>	<u>1.0</u>

3.4.1 Load Factors and Load Combinations

Revise the last Paragraph as follows:

The load factor for live load in Extreme Event Load Combination I, γ_{EQ} , shall be determined on a project-specific basis for operationally important structures. For ordinary standard bridges $\gamma_{EQ} = 0$.

Add the following after the last Paragraph:

Engineering judgment shall be exercised when applying blast loadings and when combining them with other loads.

C3.4.1

Revise the 2nd to last Paragraph as 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.~~

Add the following after the last Paragraph:

Blast loads are considered an Extreme Event case of loading. However, not enough information exists at the time of this writing (2006) to determine what other loads should be combined with blast loads and the appropriate load factors.

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3.6.1.2.6 *Distribution of Wheel Loads Through Earth Fills*

Revise the 2nd Paragraph as follows:

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft. or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by ~~either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases.~~ The provisions of Articles ~~3.6.1.1.2 and 3.6.1.3~~ shall apply.

C3.6.1.2.6

Add the following after the 2nd Paragraph:

Select granular backfill is not used for embankments in California.

The multiple presence factor should not be applied when designing the top slab of culverts.

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3.6.1.3.1 General

Add a 4th Bullet 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 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.~~

C3.6.1.3.3

Add the following after the 4th Paragraph:

The force effects due to one 32-k 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-k axles spaced 4'-0" on center, (design tandem). Also, the 54-k 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

C3.6.1.4.1

Revise the 1st Paragraph as follows:

Add 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.

For the Fatigue II limit state, the fatigue load shall be one Permit truck as specified in Figure 1.

The fatigue Permit Truck specified in Figure 1 represents the majority of permit trucks allowed in California.

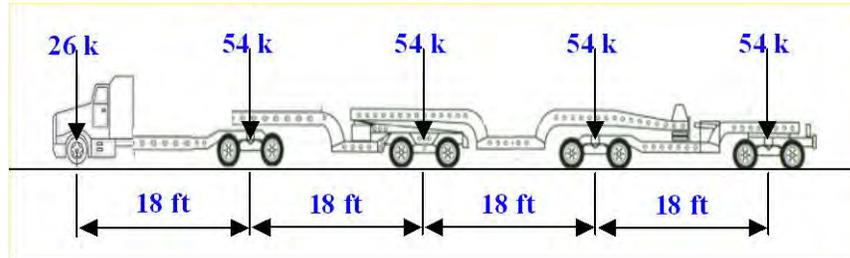


Figure 3.6.1.4.1-1 Fatigue Permit Truck

3.6.1.4.2 Frequency

C3.6.1.4.2

Add the following after the last Paragraph:

Add the following after the last Paragraph:

In the absence of specific data, *ADTT* should be taken as 2500 and 20, for the Fatigue I limit state and the Fatigue II limit state, respectively.

ADTT of 2500 for the HS-20 fatigue truck has been successfully used for design of new structures and widenings in California. Based on the variation of sizes, weight and volumes of P5 through P13 Permit trucks operating in California, along with the growth rate of 1% within the 75-year design life; the volumes of P5 through P13 trucks are conservatively converted to an equivalent fatigue P9 permit truck with an *ADTT* = 20.

3.6.1.6 Pedestrian Loads

Revise the 1st and 2nd Paragraphs as follows:

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. For a sidewalk that can be removed in the future, the vehicular live loads shall be applied at 1 ft from edge-of-deck for design of the overhang, and 2 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 ~~for a live load of 0.085 ksf~~ in accordance with AASHTO's *LRFD Guide Specifications for the Design of Pedestrian Bridges*.

C3.6.1.6

Delete the 2nd Paragraph as follows:

~~The conservatism in this Article reflects the unpredictable nature of pedestrian load, which gains significance where it becomes a primary load.~~

3.6.1.6 Pedestrian Loads

Delete the 3rd and 4th Paragraph:

~~Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance, and/or other incidental vehicles, these loads shall be considered in the design. The dynamic load allowance need not be considered for these vehicles.~~

~~Where vehicles can mount the sidewalk, sidewalk pedestrian load shall not be considered concurrently.~~

Add the following article 3.6.1.8:

3.6.1.8 Permit Vehicles

3.6.1.8.1 General

C3.6.1.6

Delete the 3rd Paragraph:

~~Snow removal and other maintenance vehicles sometimes have access to pedestrian bridges. The slow speed of such vehicles justifies the omission of dynamic effects.~~

C3.6.1.8.1

Add the following:

Permit design live loads, or P loads, are special design vehicular loads. The weights and spacings of axles and wheels for the overload truck shall be as specified in Figure 3.6.1.8.1-1.

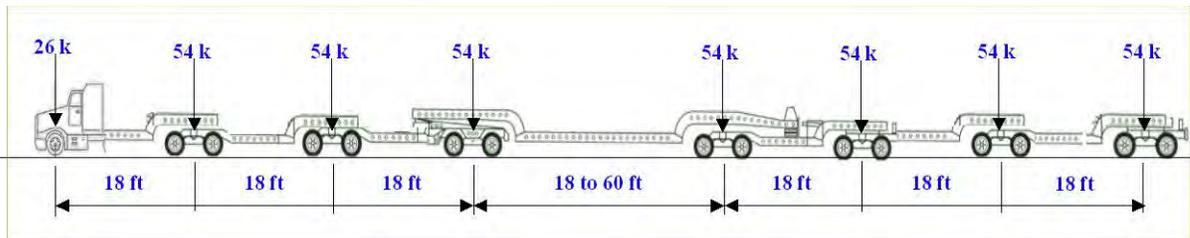


Figure 3.6.1.8.1-1 California P15 truck

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. However, when only one lane of permit is being considered, the MPF for one loaded lane shall be 1.0.

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 1 for dynamic load allowance....

Revise Table 3.6.2.1-1 as follows:

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

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

C3.6.2.1

Revise the 4th Paragraph 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 1 is the product of 4/3 and the basic 25 percent. California removed the 4/3 factor for Strength II because a 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 HL93. The HL93 tandem*1.33 + lane generally has a greater force effect than that due to the P15 on short-span bridges.

C3.6.2.1

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.

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3.6.3 Centrifugal Forces: CE**C3.6.3**

Revise the 1st Paragraph 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, design or tandem, or permit vehicle and the factor *C*, taken as:

Revise the 2nd Paragraph as follows:

Highway design speed shall not be taken to be less than the value specified in the current edition of the AASHTO publication, *A Policy of Geometric Design of Highways and Streets*, the Caltrans Highway Design Manual (current edition), or as otherwise directed. The design speed for permit vehicles shall be 25 mph, maximum.

Revise the 4th Paragraph as follows:

Centrifugal forces may ~~shall~~ be applied horizontally at a distance 6.0 ft. above the roadway surface. A load path to carry the radial force to the substructure shall be provided.

Revise the 4th Paragraph 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 the 2nd Paragraph as follows:

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 ~~a distance of 6.0 ft. above~~ 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

Add a new last Paragraph 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.

3.7.5 Change in Foundations Due to Limit State for Scour

Revise 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 1.

Table 3.7.5-1 Scour Conditions for Limit State Load Combinations

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

The consequences of changes in foundation conditions resulting from the ~~design Q_{100} base 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.~~

C3.7.5

Revise the 2nd Paragraph 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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3.8.1.2.2 Loads from Superstructures

Revise as follows:

Where the wind is not taken as normal to the structure, the base wind pressures, P_B , for various angles of wind direction may be taken as specified in Table 1, and shall be applied to a single plane of exposed area. Longitudinal and transverse pressures shall be applied horizontally to the exposed surface area of the superstructure as seen in a side view. The skew angle shall be taken as measured from a perpendicular to the longitudinal axis. The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal pressures shall be applied simultaneously.

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3.8.1.3 Wind Pressure on Vehicles: WL

Revise the 1st Paragraph 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~~ continuous force of 0.10 klf acting normal to, and 6.0 ft. above the roadway and shall be transmitted to the structure.

C3.8.1.3

Add the following after the 2nd Paragraph:

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 is negligible for most applications.

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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 as follows:

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

3.12.2 Uniform Temperature

Revise as follows:

The design thermal movement associated with a uniform temperature change shall ~~may~~ 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 1. The difference between the extended lower or upper boundary and the base construction temperature assumed in the design shall be used to calculate forces due to thermal deformation effects. Force effects shall be calculated using gross section properties and the lower value for γ_{TU} .

3.12.2.1 Temperature Range for Procedure A

Revise the 2nd Paragraph as follows:

Unless otherwise specified, the minimum and maximum temperatures specified in Table 1 shall be taken as $T_{MinDesign}$ and $T_{MaxDesign}$, respectively, in Eqs. 1 and 2 ~~3.12.2.3-1~~.

Add a 3rd Paragraph as follows:

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

$$\Delta_T = \alpha L (T_{MaxDesign} - T_{BaseConstr}) \quad (\text{Eq. 3.12.2.1-1})$$

$$\Delta_T = \alpha L (T_{MaxDesign} - T_{BaseConstr}) \quad (\text{Eq. 3.12.2.1-2})$$

where:

$T_{BaseConstr}$ \equiv base construction temperature ($^{\circ}\text{F}$)

L \equiv expansion length (in.)

α \equiv coefficient of thermal expansion (in./in./ $^{\circ}\text{F}$)

3.12.2.2 Temperature Range for Procedure B

Delete contents of the entire Article including Commentary and Figures.

C3.12.2.1

Add a 4th & 5th Paragraph as follows:

Unless otherwise specified, the base construction temperature may be taken as 60 $^{\circ}\text{F}$. In no case shall the design thermal movement range for force effects, Δ_T , be investigated for a rise or fall of less than 35 $^{\circ}\text{F}$.

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).

3.12.2.3 Design Thermal Movements

Revise the 1st Paragraph as follows:

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

C3.12.2.3

Add after the 1st Paragraph as follows:

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. In calculating Δ_T for joints and bearings, Δ_T shall be multiplied by the larger value shown for γ_{TV} in Table 3.4.1-1. Refer to Section 14 for additional design requirements for joints and bearings.

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C3.12.4

Add a new last sentence as follows:

The load factor may be reduced to 1.0 if physical testing approved by the Owner is performed to establish material properties.

C3.12.5

Add a new last sentence as follows:

The load factor may be reduced to 1.0 if physical testing approved by the Owner is performed to establish material properties.

Add article 3.12.6.1

3.12.6.1 Force Effects due to Settlement: SE

1. For predefined settlements used for geotechnical design of foundations, that is 1.0" for continuous spans and 2.0" for simple spans:
 - When geotechnical information indicates that actual differential settlement is not expected to exceed 0.5", settlement does not need to be considered in the structure analysis or design.
 - When geotechnical information indicates that differential settlement is likely to exceed 0.5", 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. When the Engineer performs 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.

Add the following:

3.12.7 Secondary Forces from Post-Tensioning, PS

The application of post-tensioned prestress forces on a statically indeterminate structure, produces reactions at the structure's support and internal forces that are collectively called secondary forces.

C3.12.7

In frame analysis software, secondary forces are generally obtained by subtracting the primary prestress forces from the total prestressing.

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3.14.11 Barge Collision Force on Pier

Delete the last Paragraph.

~~The impact force for design barges larger than the standard hopper barge shall be determined by increasing the standard hopper barge impact force by the ratio of the larger barge's width to the width of the standard hopper barge.~~

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REFERENCES

Add References:

AASHTO. 2009. *Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges*, Second Edition, GSVCB-2. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Guide Specifications for Design of Pedestrian Bridges*, Second Edition, GSDPB-2. American Association of State Highway and Transportation Officials, Washington, DC.

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