Chapter 12

Structures

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Acronyms

Authority	California High-Speed Rail Authority
CWR	Continuous Welded Rail
HST	High-Speed Train
LRFD	Load and Resistance Factor Design
OCS	Overhead Contact System
RLD	Relative Longitudinal Displacement
RVD	Relative Vertical Displacement
REJ	Rail Expansion Joints
SEJ	Structural Expansion Joints
TCL	Track Centerline
TOR	Top of Rail
VTSI	Vehicle-Track-Structure Interaction

Note: Additional Acronyms are found in Section 12.2 and Table 12-5 of this chapter.

12 Structures

12.1 Scope

This chapter provides design criteria for structures supporting California High-Speed Train (HST) service including but not limited to bridges, aerial structures, grade separations, earth retaining structures, cut-and-cover underground structures, station structures, surface facilities and buildings.

12.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards. Design shall meet applicable portions of the general laws and regulations of the State of California and of respective local authorities. Facilities shall be designed in accordance with applicable portions of the following standards and codes:

- American Concrete Institute (ACI)
  - ACI 318: Building Code Requirements for Reinforced Concrete
  - ACI 350: Code Requirements for Environmental Engineering Concrete Structures and Commentary
- American Welding Society (AWS)
  - AWS D1.1/D1.1M Structural Welding Code-Steel
  - AWS D1.8/D1.8M Structural Welding Code-Seismic Supplement
- American Association of State Highway and Transportation Officials (AASHTO)/AWS D1.5M/D1.5 Bridge Welding Code
- California Building Code (CBC)
- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering
- American Institute of Steel Construction (AISC), Steel Construction Manual,
- American Society of Civil Engineers (ASCE) 7-05; Minimum Design Loads for Buildings and Other Structures
- California Occupational Safety and Health Administration (Cal/OSHA) Department of Industrial Relations
- California Department of Transportation (Caltrans) Bridge Design Manuals

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  - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification with California Amendments
  - Bridge Memo to Designers Manual (CMTD)
  - Bridge Design Practices Manual (CBPD)
  - Bridge Design Aids Manual (CBDA)
  - Bridge Design Details Manual (CBDD)
  - Seismic Design Criteria (CSDC)
  - Office of Special Funded Projects (OSFP) Information and Procedures Guide
- Code of Federal Regulations (CFR)
- United States Department of Transportation Federal Highway Administration; Technical Manual for Design and Construction of Road Tunnels – Civil Elements; Publication No. FHWA-NHI-09-010

Other international standards are used in the development of these criteria, including the following:

- European Standard EN 1991-2:2003 Actions on Structures – Part 2: Traffic Loads on Bridges
- European Standard EN 1990:2002 +A1 Basis of Structural Design annex A2: Application to Bridges
- International Federation for Structural Concrete (FIB) Model Code for Concrete Structures, 1990 (For Time Dependent Behavior of Concrete)

12.3 Types of Structures

Structures supporting high-speed train service are classified as the following:

- Bridges – HST trackway structures crossing rivers, lakes, or other bodies of water
- Aerial Structures – elevated HST trackway structures including bridges, viaducts and HST grade separations
- Grade Separations – structures separating trackways from railroad, highway or pedestrian usage
- Earth Retaining Structures – including U-walls, trenches, and retaining walls
- Cut-and-Cover Underground structures – including cut-and-cover line structures
- Bored Tunnels
- Mined Tunnels

- Surface Facilities and Buildings – including station buildings, station parking structures, secondary and ancillary buildings, sound walls, and miscellaneous structures
- Underground Ventilation Structures
- Underground Passenger Stations
- Equipment and Equipment Supports

12.4 Structural Design Requirements

Structures shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with the consideration of inspectability, and maintainability, as specified in AASHTO LRFD with California Amendments unless otherwise modified here.

12.4.1 Structural Design Parameters

- Structures shall be designed for the appropriate loadings and shall comply with the HST structure gauge per the *Trackway Clearances* chapter.
- The design life for structures shall be 100 years. Elements such as expansion joints and bearings that need to be replaced in the life of the structures, the Contractor shall evaluate the life of each element and specify the replacement procedures which the element can be replaced within the non-operation hours of the HST service.
- Requirements for noise and vibration are defined in the environmental documents including materials and specific locations and measurements.
- Structural design criteria shall apply to structures adjacent to, above, or below the HST tracks and which performance could directly affect HST operation. This includes aerial structures carrying HSTs and newly constructed highway or ancillary structures which will directly affect the HST operations.
- Structures shall be designed so that the elements normally replaced during maintenance can be readily replaced with minimal impact to HST operations.
- The bridges and aerial superstructures supporting HSTs shall be designed to meet stiffness requirements in order to meet serviceability and comfort requirements for HST operation.
- Permanent and temporary structures including falsework shall be designed in accordance with the clearance requirements. Clearance requirements for falsework are only applicable where the falsework is erected over an operational road or railway.
- Design of structures shall consider loads and effects due to erection equipment, construction methods, and sequence of construction.
- Design and construction of HST facilities shall comply with the approved and permitted environmental documents.

- Only non-flammable materials are allowed in construction. Timber is allowed in construction of falsework.

12.4.2 Seismic Design

For definition of structural classification as Primary or Secondary structures, refer to the *Seismic* chapter. For seismic design criteria for Primary and Secondary structures, as defined in the *Seismic* chapter, refer to the *Seismic* chapter.

12.5 Permanent and Transient Loads and Load Factors for Structures Supporting HST

This section specifies the loads and forces, load factors and load combinations for the application of permanent and transient loads for structures directly supporting HST. This section defines loads specific to bridges, aerial structures, and grade separations. These loads are applicable to earth retaining structures and cut-and-cover structures.

Facility loads, such as those for buildings and stations not supporting high-speed trains, are specified in Section 12.7 - Structural Design of Surface Facilities and Buildings.

For structures carrying highway loads, AASHTO LRFD with Caltrans Amendments shall apply with supplementary provisions herein to account for loads or seismic performance criteria specific to HST operations.

The dynamic analyses to determine dynamic impact factors and ensure the passenger comfort associated with HST rolling stock loadings and the requirements of the track-structure interaction are defined in Section 12.6 - Track-Structure Interaction.

12.5.1 Permanent Loads

12.5.1.1 Dead Load (DC, DW)

The dead load shall include the weight of structure components, appurtenances, utilities attached to the structure, earth cover, finishes, and permanent installations such as tracks, ballast, conduits, piping, safety walkways, walls, sound walls, electrification and utility services.

In the absence of more precise information, the unit weights specified in Table 12-1 shall be used for dead loads.

DC refers to the dead load of structural components and permanent attachments supported by the structure including, tracks, ballast, plinths, cable troughs, parapet walls, sound walls, overhead contact system (OCS), etc.

DW refers to the dead load of non-structural attachments which are permanent or non-permanent attachments including, utilities, cables, finishes, etc.

If applicable, dead load shall be applied in stages to represent the sequence required to construct the structure. Analysis shall consider the effect of the maximum and minimum loading imposed on the structure during construction or resulting from placement or removal of earth cover.

Table 12-1: Unit Weight of Common Materials

Item	Unit Weight	Reference
Electrification (OCS and fastenings)	100 pounds per foot of track	CHSTP (see note 2)
OCS poles and support	See Section 12.5.3.1	CHSTP
Cable trough including walkway surface without OCS pole	1400 plf each	CHSTP
Ballast	140 pcf	AASHTO LRFD with Caltrans Amendments
Ballasted track not including rail and fastener systems	3800 pounds per foot per track, including ties, (add 1000 plf in superelevated zones)	CHSTP
Parapet wall	800 pounds per foot each side	CHSTP
Rails and fasteners (no ties) including special trackwork	200 pounds per foot of track	AREMA
Non-ballasted track & non-ballasted track base not including rail and fastener systems	2500 pounds per foot per track, (add 1000 plf in superelevated zones)	CHSTP
Soils	See Geotechnical reports described in the *Geotechnical* chapter	—
Sound wall material (clear, 1 inch thick)	65 pounds per foot for 14-foot height from TOR placed above concrete parapet	CHSTP
Systems cables in trough	200 pounds per foot of track	CHSTP

Notes:
1. For materials not listed, see AASHTO LRFD with Caltrans Amendments.
2. CHSTP refers to the weights of internal systems requirements necessary for HST operations.

12.5.1.2 Downdrag Force (DD)

Possible development of downdrag on piles or shafts shall be considered. Recommended negative skin friction values shall be as provided for the particular site in the Geotechnical reports described in the *Geotechnical* chapter or as a minimum see AASHTO LRFD with Caltrans Amendments Article 3.11.8.

12.5.1.3 Earth Pressure (EV, EH)

Substructure elements shall be proportioned to withstand earth pressure. Recommended soil parameters, earth pressures and loads due to surcharges shall be as provided for the particular site in the Geotechnical reports described in the *Geotechnical* chapter. See the *Geotechnical* chapter for determination of pressures acting on earth retaining structures. If site specific design data is not available, use AASHTO LRFD with Caltrans Amendments Article 3.11.

A. Vertical Earth Pressure (EV)

Depth of cover shall be measured from the ground surface or roadway crown, or from the street grade, whichever is higher, to the top of the underground structure. Saturated densities of soils shall be used to determine the vertical earth pressure. Recommended values given in the *Geotechnical* chapter shall be used.

B. Lateral Static Earth Pressure (EH)

For structures retaining draining cohesionless (granular) soil, lateral earth pressure shall be determined in accordance with the *Geotechnical* chapter. For structures retaining other soil types, the lateral soil pressure shall be in accordance with the recommendations specified in the *Geotechnical* chapter.

C. Lateral Seismic Earth Pressure

For increases in earth pressures caused by seismic actions see the *Geotechnical* chapter.

12.5.1.4 Earth Surcharge (ES)

Surcharge loads (ES) are vertical or lateral loads resulting from loads applied at or below the adjacent ground surface. Procedures for determining surcharge load shall be as given in the Geotechnical reports described in the *Geotechnical* chapter.

12.5.1.5 Earth Settlement Effects (SE)

Earth settlement effects (SE) are forces or displacements imposed on a structure due to either uniform or differential settlement under sustained loading. Recommended values of settlement as given in the Geotechnical Reports described in the *Geotechnical* chapter shall be used.

Tolerable settlement on foundations shall be developed by the designer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on structural and operational performance. Operational settlement limits are listed in the *Geotechnical* chapter.

At and near water crossings, scour potential shall also be considered for earth settlement effects.

12.5.1.6 Creep Effects (CR)

For the effects due to creep of concrete (CR), the requirements in AASHTO LRFD with Caltrans Amendments Article 5 shall be used.

12.5.1.7 Shrinkage Effects (SH)

For the effects due to shrinkage of concrete (SH), the requirements in AASHTO LRFD with Caltrans Amendments Article 5 shall be used.

12.5.1.8 Secondary Forces from Prestressing (PS)

Secondary forces from prestressing (PS) shall be accounted for in the design. Such secondary forces arise during prestress of statically indeterminate structures, which produce additional internal forces and support reactions.

12.5.1.9 Locked-in Construction Forces (EL)

Miscellaneous locked-in construction force effects (EL) resulting from the construction process shall be considered. Such effects include jacking apart adjacent cantilevers during segmental construction.

12.5.1.10 Water Loads (WA)

The effects of ground water hydrostatic force, including static pressure of water, buoyancy, stream pressure, and wave loads (WA) shall be considered using the requirements in AASHTO LRFD with Caltrans Amendments Article 3.7. Recommended values given in the Geotechnical Reports described in the *Geotechnical* chapter shall be used. Refer to Section 12.11.2.7 for Hydrostatic pressure for trench and cut-and-cover structures.

Adequate resistance to flotation shall be provided at sections for full uplift pressure on the structure foundation, based upon the maximum of either the maximum probable height of the water table defined in the Geotechnical Reports described in the *Geotechnical* chapter or the maximum flood condition based on the drainage requirements in the *Drainage* chapter. For the completed structure, uplift resistance shall consist of the dead weight of the completed structure and the weight of backfill overlying the structure (within vertical planes drawn through the outer edges of the structure roof and through the joints).

Hydrostatic pressure shall be applied normal to surfaces in contact with groundwater with a magnitude based on the height of water table and the applicable water density.

The change in foundation condition due to scour shall be investigated per AASHTO LRFD with Caltrans Amendments Article 3.7.5.

12.5.2 Transient Loads

12.5.2.1 Live Loads (LLP, LLV, LLRR, LLH, LLS)

Live loads are due to high-speed trains, other trains (not HST), Amtrak, passenger rail, shared-use rail trains, highway loads, construction equipment, and pedestrians.

A. <u>Floor, Roof, and Pedestrian Live Loads (LLP)</u>

For the force effects due to floor and roof live loads (LLP), refer to Section 12.7 - Structural Design of Surface Facilities and Buildings. Section 12.7 includes provisions for aerial trackway supporting service walkways.

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B. High-Speed Train Live Loads (LLV)

The project specific rolling stock has not yet been determined. Once the rolling stock is determined, the live load criteria will be included. Trainsets similar to those being considered are presented in Section 12.6.

C. Shared-Use Track Train Live Loads (LLRR)

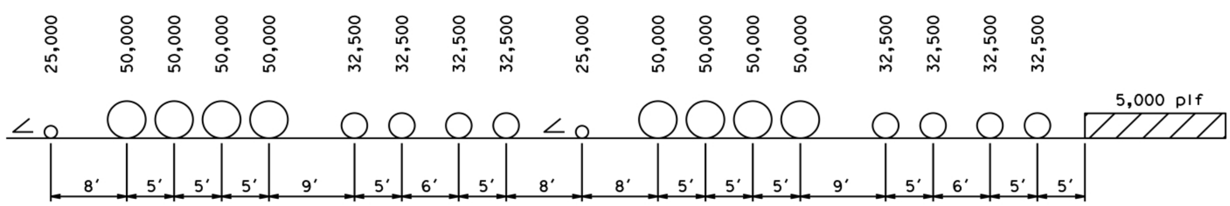
Structures that will support shared service with another railroad system such as AMTRAK, Caltrain, Metrolink, UPRR, BNSF, etc., have specific criteria that must be followed in addition to the requirements provided herein for high-speed trains.

AMTRAK loading is describes in Section 12.5.2.1-E below. Additionally, design shall meet the requirements described in the *Seismic* chapter and Section 12.6 - Track-Structure Interaction.

D. Maintenance and Construction Train Live Loads: Cooper E-50 Loading (LLRR)

Structures shall be designed to support maintenance and construction trains, vertical loads are defined as the Cooper E-50 in the AREMA Specification, see Figure 12-1.

Figure 12-1: Cooper E-50 Loading (LLRR)



For the case of multiple tracks on the bridge, LLRR shall be as follows:

- For two tracks, full live load on two tracks.
- For three tracks, full live load on two tracks and one-half on the other track.
- For four tracks, full live load on two tracks, one-half on one track, and one-quarter on the remaining one.
- For more than four tracks, to be considered on an individual basis.

The tracks selected for full live load shall be those tracks which will produce the most critical design condition on the member under consideration.

E. Amtrak Live Loads

Designated segments of the HST alignment are required to be designed to provide for Amtrak service. These segments shall be designed to support Cooper E-50 loads as described in the AREMA Manual. These structure segments shall also be designed to meet the requirements for structures supporting HSTs, including but not limited to track-structure compatibility and the seismic requirements.

These structures shall be designed to resist two axles weighting 75 kips each with a longitudinal spacing of 9.0 feet. This additional loading is required to account for local effects of Amtrak locomotives.

F. Highway Live Loads (LLH)

Facilities required to support highway loads over HST shall be designed to the requirements of AASHTO LRFD with Caltrans Amendments Article 3.6.1. For facilities intended to support highway permit loads, Caltrans guidelines shall be followed for the routing and sizes of the permit vehicles.

G. Live Load Surcharge (LLS)

An area live load surcharge (LLS) shall be applied at the ground surface both over and adjacent to underground structures, as applicable, to account for presence of live load. Live load surcharge results from presence of LLRR, LLV, LLH, possible future roadways, sidewalk live loads, or construction live loads.

Methods for lateral distribution of live load surcharge due to rail loading shall be in accordance with AREMA. Lateral distribution of highway surcharge shall be in accordance with AASHTO LRFD with Caltrans Amendments Article 3.11.6.4.

No impact factors apply to LLS for walls. A reduction of impact for buried components shall be applicable as specified in AASHTO LRFD with Caltrans Amendments Article 3.6.2, with the 33 percent base impact value modified as applicable to LLRR or LLV, as given herein.

Recommended coefficients for lateral surcharge loading shall be as recommended in the Geotechnical reports described in the *Geotechnical* chapter.

H. Live Loading for Fatigue Assessment

For structures carrying high-speed trains, the project specific rolling stock (LLV) plus impact (I) shall be used for fatigue assessment of structures. Various trainsets LLV, that may be selected are described in Section 12.6.6.1 and dynamic impact shall be determined according to the requirements in Section 12.6.6.3. The methods of AASHTO LRFD with Caltrans Amendments Article 3.6.1.4 shall be used to evaluate fatigue loads.

The fatigue assessment shall be performed for structural elements which are subjected to fluctuations of stress. For structures supporting multiple tracks the loading shall be applied to a minimum of 2 tracks in the most unfavorable positions. The fatigue damage shall be assessed over the required structural life of the structure. During maximum service condition there are 2.8 million axle loads per track per year.

12.5.2.2 Vertical Impact Effect (I)

Moving trains and vehicles impart dynamic loads to bridges, which are considered through a dynamic coefficient or impact factor. The static effects of the design train loads, other than centrifugal, traction, braking, nosing and hunting shall be increased by the percentages specified.

Dynamic analysis is required for structures carrying HSTs (LLV) in order to determine impact effects. This is addressed in detail within Section 12.6 –Track-Structure Interaction.

For determining impact factors (I) associated with maintenance and construction train loading (LLRR) on ballasted track, AREMA Specifications shall be used as follows:

Ballasted track:

- Reinforced or prestressed concrete bridges:

$I = 60\%$ where L $\leq$ 14 feet

$I = \dfrac{225}{\sqrt{L}}$ where 14 feet < L $\leq$ 127 feet

$I = 20\%$ where L > 127 feet

- Steel bridges:

$I = 40 - \dfrac{3L^2}{1600}$ where L < 80 feet

$I = 16 + \dfrac{600}{L - 30}$ where L $\geq$ 80 feet

L = span length

For determining impact factors (I) associated with maintenance and construction train loading (LLRR) for direct fixation on concrete non-ballasted track with spans less than 40 feet, European Standard EN 1991-2 shall be used as modified below. For spans longer than 40 feet, AREMA ballasted track impact factors shall be used.

Direct fixation on concrete non-ballasted track:

$$I = 100\left(\frac{2.16}{\sqrt{0.305L} - 0.2} - 0.27\right) \leq 100\% \text{ where L} \leq 40 \text{ feet}$$

L = Span length for member under consideration (main girder, bridge deck, etc.)

The calculated value shall be applied at top of rail (TOR) as a percentage of live load.

An additional ±20 percent imbalance of live load shall be applied to each rail as a vertical force to model the couple caused by potential rocking of the train. The couple shall be applied on each track in the direction which will produce the most unfavorable effect in the member under consideration.

For determining impact factors (I) associated with highway loading (LLH), AASHTO LRFD with Caltrans Amendments dynamic load allowance, IM as defined in AASHTO LRFD with Caltrans Amendments shall be used.

Impact effect applies to the following:

- Superstructure, including steel or concrete supporting columns, steel towers, legs of rigid frames, and generally those portions of the structure which extend down to the main foundation.
- The portion above the ground line of concrete or steel piles that support the superstructure directly.

Impact effect does not apply to the following:

- Retaining walls, wall-type piers, and piles except those described above.
- Foundations and footings entirely below ground, and base slabs which are in direct contact with earth.
- Floor, roof, and pedestrian live loads (LLP).

12.5.2.3 Centrifugal Force (CF)

For tracks on a curve, centrifugal force (CF) shall be considered as a horizontal load applied toward the outside of the curve. Multiple presence factors shall apply to centrifugal forces. See the *Track Geometry* chapter for the range of radius values.

For centrifugal forces from carrying vehicular traffic, refer to AASHTO LRFD with Caltrans Amendments.

The centrifugal force (CF) is a function of the train live load (LLRR or LLV), speed, and horizontal radius of curvature:

For LLRR, use AREMA requirements

For LLV, CF acts at 6 feet above TOR

$CF = (LLRR \text{ or } LLV) \times [0.0668 * V^2 * f / R]$

Where:

V = train speed (mph)

R = horizontal radius of curvature (feet)

f = reduction factor, not to be taken less than 0.35:

$f = 1$, for LLRR, for $V \leq 75$ mph

$f = 1 - [(V - 75)/621.4] \times [506/V + 1.75] \times [1 - (9.45/L)^{1/2}] \geq 0.35$, for LLRR, V > 75 mph

f = 1, for LLV, all speeds

L = length in feet of the loaded portion of curved track on the bridge.

If the maximum line speed at the site is in excess of 75 mph, the centrifugal force shall be investigated at 75 mph with a reduction factor of 1.0, and at the maximum line speed with a reduction factor less than 1.0.

The effect of superelevation shall be considered when present. The superelevation effect shifts the centroid of the train laterally producing an unequal transverse distribution between rails. Consideration shall be given to the cases in which the train is moving and at rest condition.

12.5.2.4 Traction and Braking Forces (LF)

A. LLRR

For traction and braking forces (LF) from passenger trains, freight trains, maintenance and construction trains (LLRR) taken from AREMA Section 2.2.3:

Traction force = $N(25\sqrt{L})$ kips, acting 3 feet above TOR

Braking force = $N(45 + 1.2L)$ kips, acting 8 feet above TOR

Where:

L = length in feet of portion of bridge under consideration

N = ratio of Cooper train load to Cooper E80 loading for the sizes of trains that will use the structure (i.e. for Cooper E50, N = 0.625)

The LF loads for LLRR are to be distributed over the length of portion of bridge under consideration up to the maximum length of train. Multiple presence factors shall apply.

B. LLV

For traction and braking forces (LF) from high-speed trains (LLV) taken from European Standard Eurocode EN 1991-2, Article 6.5.3:

Traction force = 2.26 kips per linear foot or 25 percent of train load (if known), with a maximum value of 225 kips, acting at TOR

Braking force = 1.37 kips per linear foot or 25 percent of train load (if known), with a maximum value of 1350 kips, acting at TOR

Traction and braking forces will be reviewed and confirmed when the rolling stock is selected.

C. <u>LLH</u>

For braking forces (LF) from highway loading (LLH), AASHTO LRFD with Caltrans Amendments Article 3.6.4 shall be used.

12.5.2.5 Nosing and Hunting Effects (NE)

For structures with non-ballasted track and direct fixation fasteners, nosing and hunting effects (NE) of the wheels contacting the rails shall be accounted by a 22 kip horizontal force applied to the top of the low rail, perpendicular to the Track Centerline (TCL) at the most unfavorable position.

NE is not applicable for the design of bridge decks with ballasted track. NE is not applicable to LLRR and LLH.

NE shall be applied simultaneously with centrifugal force (CF).

12.5.2.6 Wind Loads (WS, WL)

Wind Load on Structures (WS) and Wind Load on Trains (WL) shall be calculated in accordance with requirements in AASHTO LRFD with Caltrans Amendments Article 3.8 with the following modifications:

- The effective wind area shall include the exposed area of all bridge elements, OCS poles, and catenary. For parapets and barriers, shielding of downwind elements from those upwind shall not be considered (i.e., the exposed area shall include the summation of parapets on the bridge).
- The base lateral load for Wind Load on Vehicles (WL) shall be revised to 0.300 klf perpendicular to the train acting 8 feet above the TOR. AASHTO LRFD with Caltrans Amendments Table 3.8.1.3-1 Wind Components on Live Load for skewed angles of incidence shall be revised proportionally to reflect the modified base lateral load.
- For structures that utilize sound walls or wind walls capable of effectively shielding the train from wind loading, consideration may be given to a reduction of WL. The reduction may be taken as the fractional height of train that is shielded by the wall. This reduction shall not exceed 50 percent of WL.

Local design elements such as parapets or components on structures shall be designed to wind loading and slipstream effects. Wind loading shall be calculated per CBC. The wind importance factor shall equal 1.15.

Wind loading for non-conventional bridge types or long-spans will require special attention (e.g., dynamic effects).

Wind loads (WS) on building and station structures are detailed in Section 12.7 – Structural Design of Surface Facilities and Buildings.

Wind loads (WS, WL) on highway structures shall be per AASHTO LRFD with Caltrans Amendments.

12.5.2.7 Slipstream Effects (SS)

A. Aerodynamic Actions from Passing Trains

The passing of high-speed trains subjects structures situated near the track to transient pressure waves. This action may be approximated by equivalent loads acting at the front and rear of the train.

Aerodynamic actions from passing trains shall be taken into account when designing structures adjacent to railway tracks.

The passing of rail traffic subjects any structure situated near the track to a traveling wave of alternating pressure and suction (see Figures 12-2 to 12-7). The magnitude of the action depends mainly on:

- Square of the speed of the train
- Aerodynamic shape of the train
- Shape of the structure
- Position of the structure, particularly the clearance between the vehicle and the structure

The actions may be approximated by equivalent loads at the ends of a train when checking ultimate and serviceability limit states and fatigue. Equivalent loads are given in Sections 12.5.2.7-B to 12.5.2.7-G.

In Sections 12.5.2.7-B to 12.5.2.7-G, the Maximum Design Speed V [mph] shall be taken as the Maximum Line Speed at the site.

At the start and end of structures adjacent to the tracks, for a length of 16.4 feet from the start and end of the structure measured parallel to the tracks, the equivalent loads in Sections 12.5.2.7-B to 12.5.2.7-G shall be multiplied by a dynamic amplification factor of 2.0.

Simple is defined here as smooth, without projections, ribs, or other obstruction.

For aerodynamic actions inside of tunnels, see the *Tunnels* chapter.

Note: For dynamically sensitive structures, the dynamic amplification factor may be insufficient and may need to be determined by a special study. The study shall take into account dynamic characteristics of the structure including support and end conditions, speed of the adjacent rail traffic and associated aerodynamic actions, and the dynamic response of the structure including the speed of a deflection wave induced in the structure. In addition, for dynamically sensitive structures a dynamic amplification factor may be necessary for parts of the structure between the start and end of the structure.

B. Simple Vertical Surfaces Parallel to the Track

Equivalent loads, ± q_{1k}, are given in Figure 12-2 and Figure 12-3.

Figure 12-2: Equivalent Loads q_{1k} for Simple Vertical Surfaces Parallel to the Track for Speeds Less than 125 mph

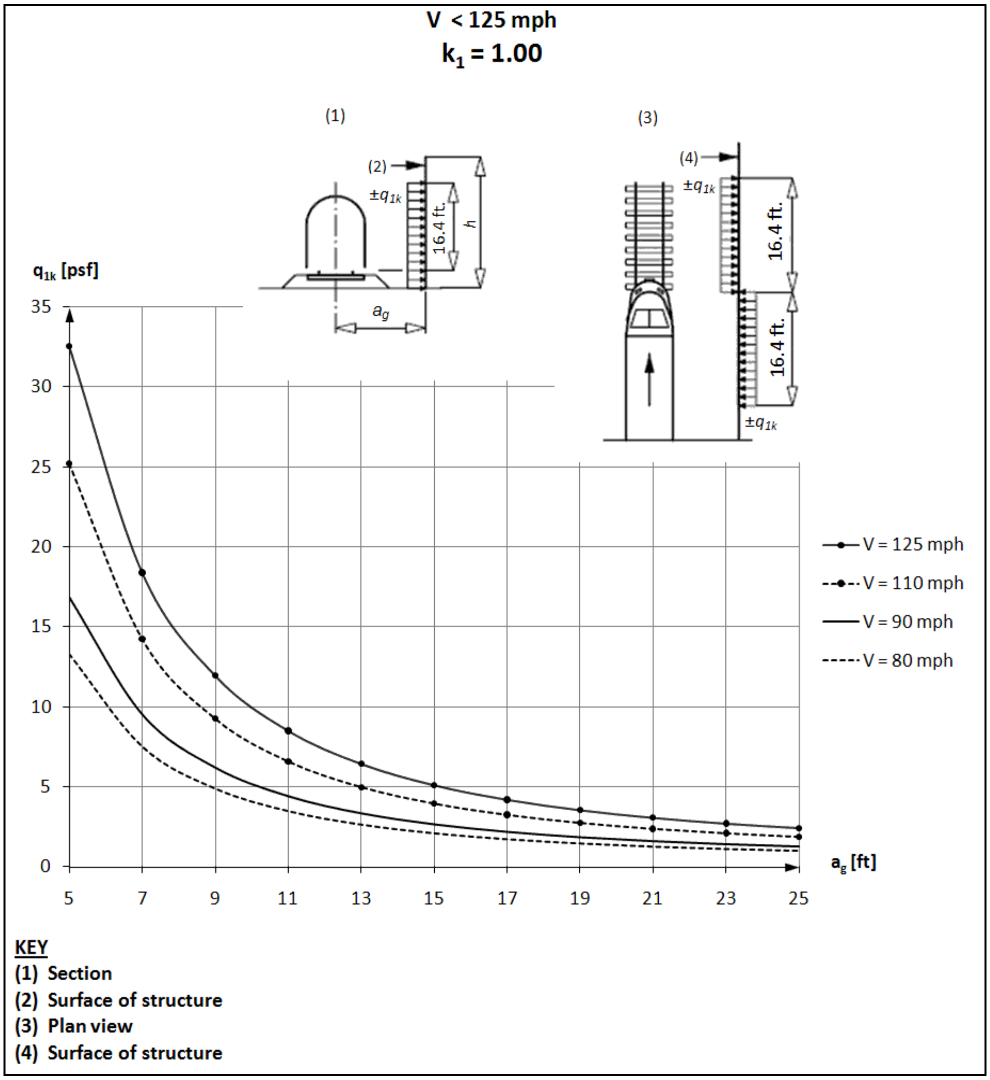
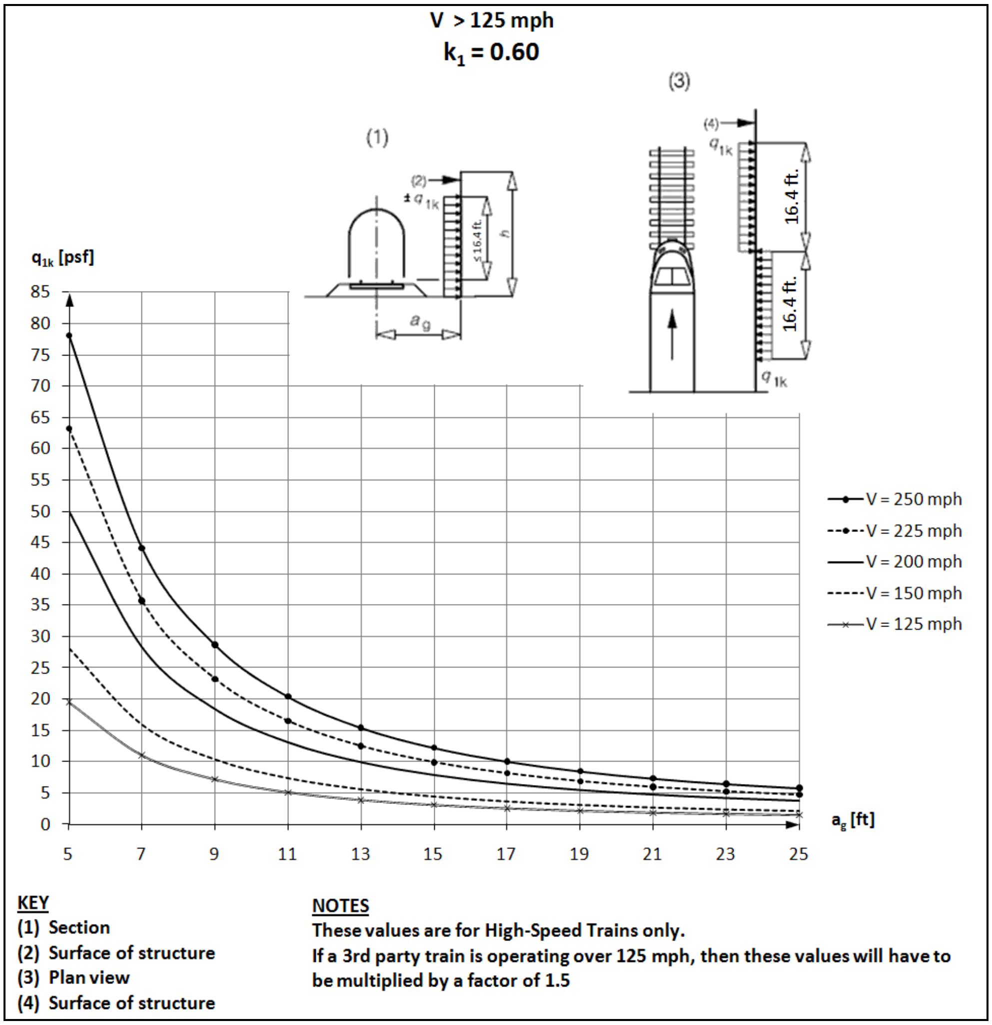


Figure 12-3: Equivalent Loads q_{1k} for Simple Vertical Surfaces Parallel to Track for Speeds Greater than 125 mph



The equivalent loads apply to trains with an unfavorable aerodynamic shape and may be reduced by:

- A factor k_1 = 0.85 for trains with smooth sided rolling stock
- A factor k_1 = 0.6 for streamlined rolling stock (*e.g.* ETR, ICE, TGV, Eurostar or similar)

If a small part of a wall with a height $\leq$ 3.00 feet and a length $\leq$ 8 feet is considered (e.g., an element of a wall), the actions q_{1k} shall be increased by a factor k_2 = 1.3.

For surfaces perpendicular to the train, the actions q_{1k} shall be taken from Figure 12-2 and Figure 12-3 for the distance indicated from TCL modified as described in the previous items.

C. Simple Horizontal Surfaces Above the Track (e.g. overhead protective structures)

Equivalent loads, ± q_{2k}, are given in Figure 12-4 and Figure 12-5.

The loaded width for the overhead structural member extends up to 33 feet to either side from the TCL.

Figure 12-4: Equivalent Loads q_{2k} for Simple Horizontal Surfaces Above Track for Speeds Less than 125 mph

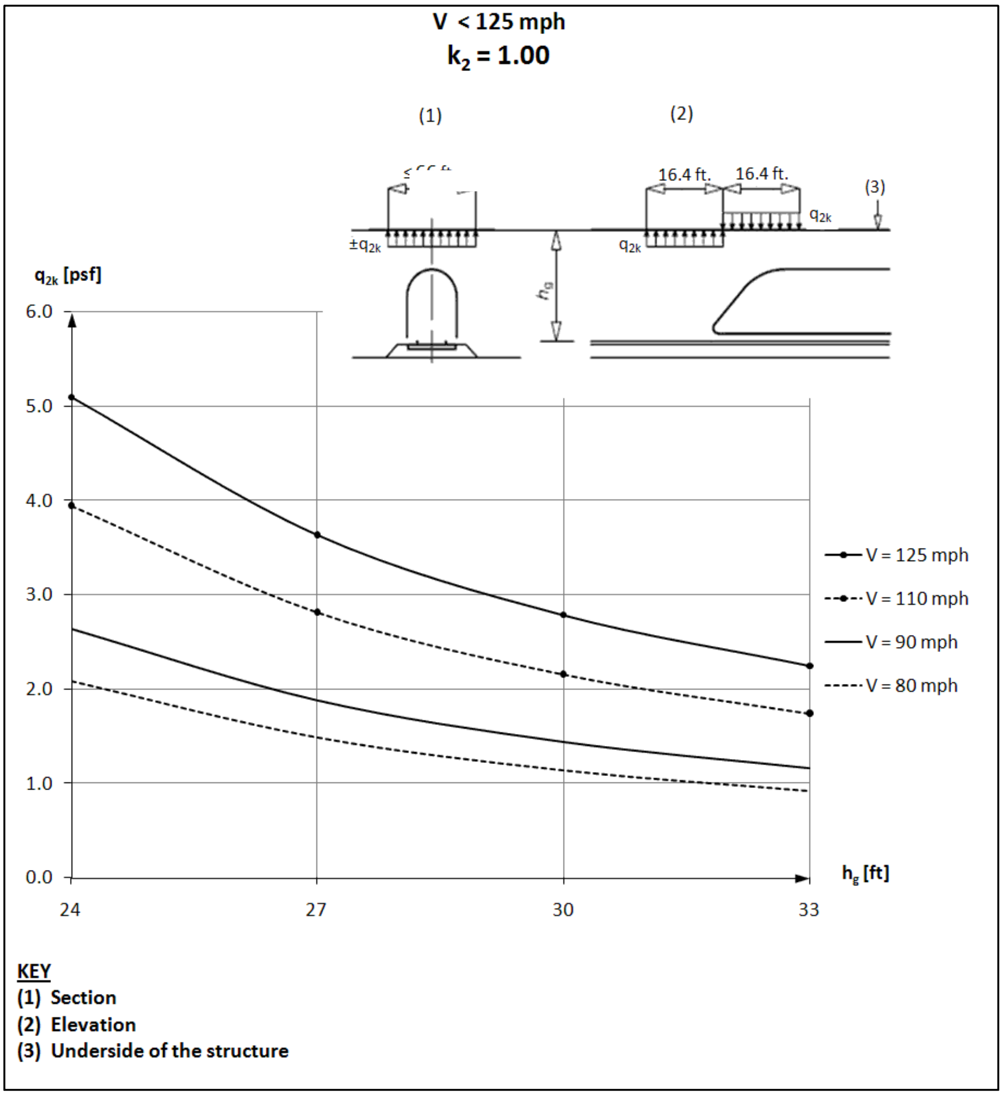
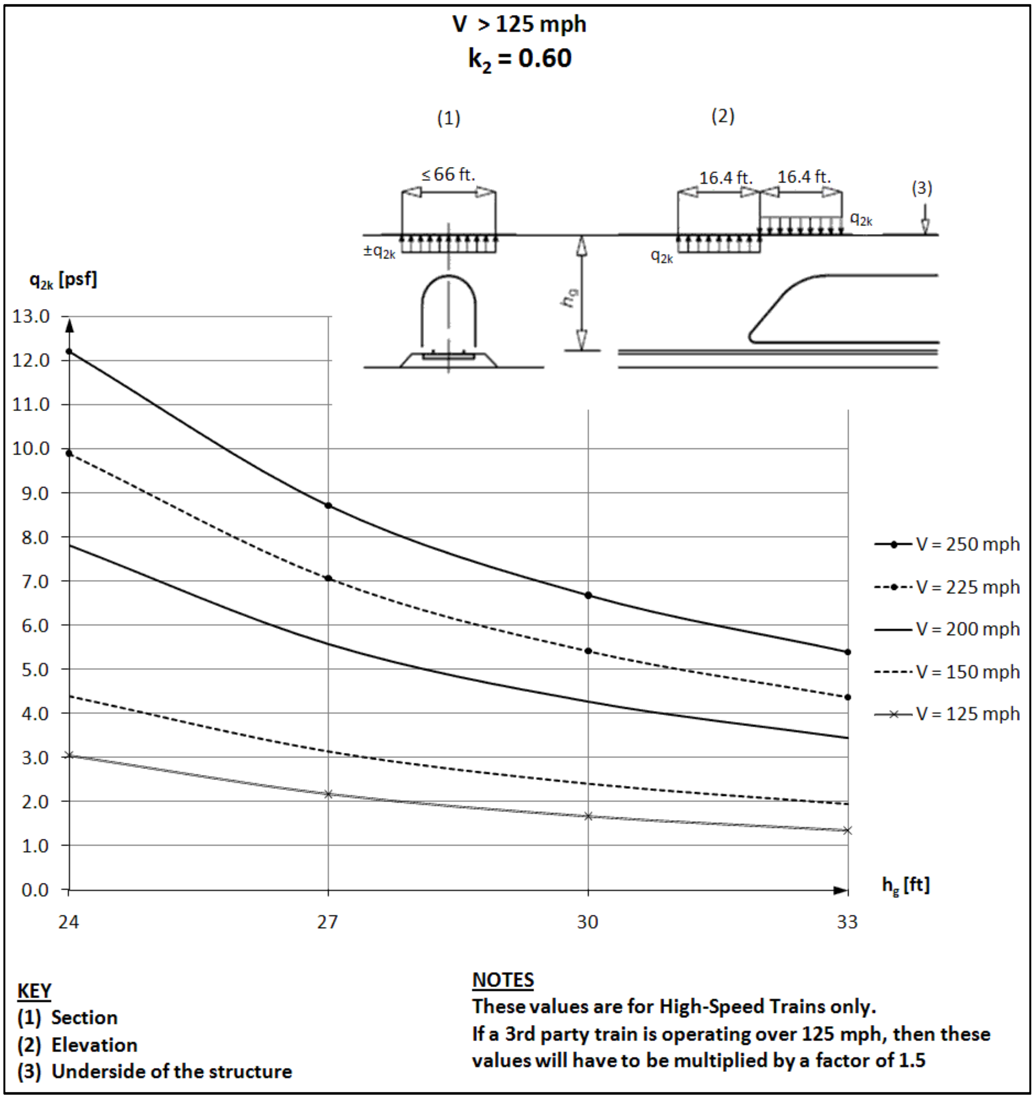


Figure 12-5: Equivalent Loads q_{2k} for Simple Horizontal Surfaces Above Track for Speeds Greater than 125 mph



For trains passing in opposite directions, the actions shall be added. The loading from trains on only two tracks shall be considered.

The actions q_{2k} may be reduced by the factor k_1 as defined in Section 12.5.2.7-B.

The actions acting on the edge strips of a wide structure (greater than 33 feet) crossing the track may be multiplied by a factor of 0.75 over a width up to 16.5 feet.

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D. <u>Simple Horizontal Surfaces Adjacent to the Track (e.g., platform canopies with no vertical wall)</u>

Equivalent loads, ± q_{3k}, are given in Figure 12-6 and apply irrespective of the aerodynamic shape of the train.

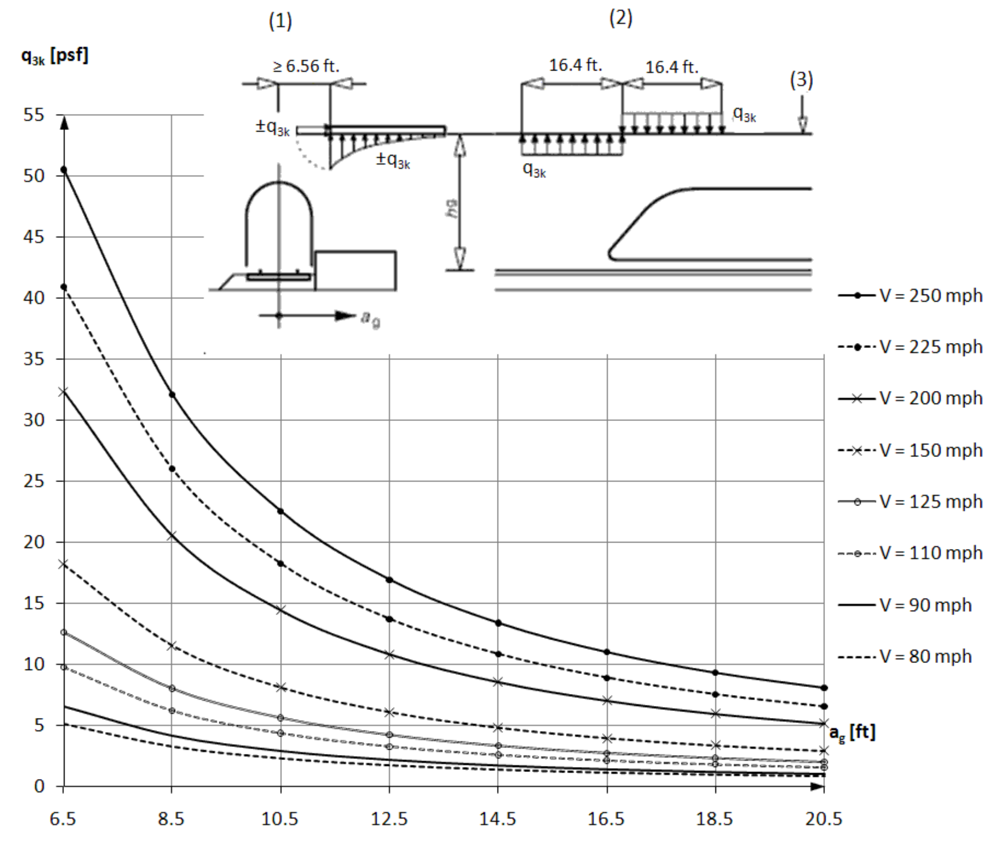
Figure 12-6: Equivalent Loads q_{3k} for Simple Horizontal Surfaces Adjacent to Track

$k_3 = \frac{(24.6 - h_g)}{12.14}$ for 12.5 ft < h_g < 24.6 ft

$k_3 = 0$ for $h_g \geq 24.6$ ft

where:

h_g distance from top of rail level to the underside of the structure.



KEY
(1) Section
(2) Elevation
(3) Underside of the structure

For every position along the structure to be designed, q_{3k} shall be determined as a function of the distance a_g from the nearest track. The actions shall be added if there are tracks on either side of the structural member under consideration.

If the distance h_g exceeds 12.5 feet the action q_{3k} may be reduced by a factor k_3.

E. Multiple-Surface Structures Alongside the Track with Vertical and Horizontal or Inclined Surfaces (e.g., noise barriers, platform canopies with vertical walls, etc.)

Equivalent loads, ± q_{4k}, as given in Figure 12-7 shall be applied normal to the surfaces considered. The actions shall be taken from the graphs in Figure 12-2 and Figure 12-3 adopting a track distance the lesser of:

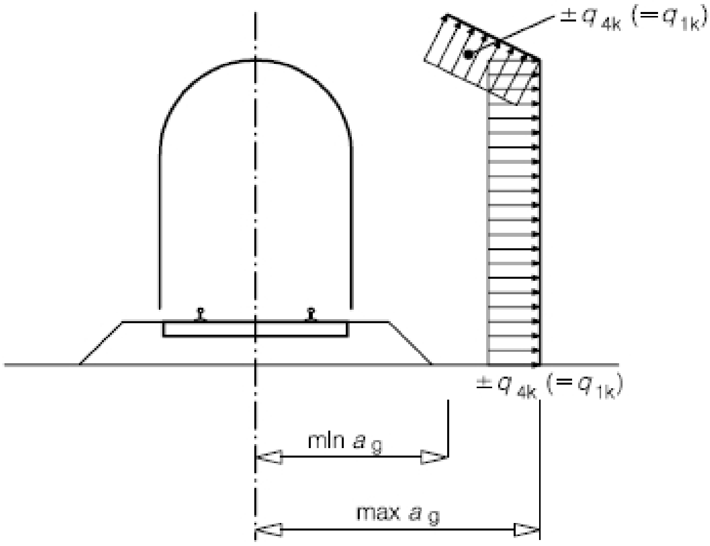
a'_g = 2.0 feet min a_g + 1.25 feet max a_g, or 20 feet

where distances min a_g and max a_g are shown in Figure 12-7

If max a_g > 20 feet the value max a_g = 20 feet shall be used

The factors k_1 and k_2 defined in Section 12.5.2.7-B shall be used

Figure 12-7: Definition of the Distances min a_g and max a_g from Centerline of Track



F. Surfaces Enclosing the Structure Gauge of the Tracks over a Limited Length (up to 65 feet) (horizontal surface above the tracks and at least one vertical wall, e.g. scaffolding, temporary construction)

Actions shall be applied irrespective of the aerodynamic shape of the train:

- To the full height of the vertical surfaces:

$\pm k_4 q_{1k}$

Where:

q_{1k} is determined according to Section 12.5.2.7-B

k_4 = 2

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- To the horizontal surfaces:

$$\pm k_5 q_{2k}$$

Where:

q_{2k} is determined according to Section 12.5.2.7-C for only one track,

k_5 = 2.5 if one track is enclosed

k_5 = 3.5 if two tracks are enclosed

G. Surfaces Perpendicular to or Crossing the Tracks over a Limited Length

Surfaces that are normal to the track such as wayside equipment and signs shall be designed to resist transient air pressure presented in Figure 12-2 and Figure 12-3 on vertical surfaces and Figure 12-4 and Figure 12-5 on horizontal surfaces.

12.5.2.8 Thermal Load (TU, TG)

For uniform (TU) and gradient (TG) temperature effects of the structure, the requirements in AASHTO LRFD with Caltrans Amendments Article 3.12 shall be used. Consideration shall be given to the maximum and minimum ambient temperatures.

12.5.2.9 Frictional Force (FR)

The force due to friction (FR) shall be established on the basis of extreme values of the friction coefficient between sliding surfaces (i.e., at bearing pads). Where appropriate, the effects of moisture, degradation, and contamination of sliding or rotating surfaces upon the friction coefficient shall be considered.

Where applicable, recommended frictional values per AASHTO LRFD with Caltrans Amendments shall be used.

12.5.2.10 Seismic Loads (MCE, OBE)

Detailed, project specific seismic design criteria are presented in the *Seismic* chapter. The *Seismic* chapter defines seismic design philosophies, seismic analysis/demand methodologies, and structural capacity evaluation procedures for the two levels of design earthquakes.

Primary structures shall comply with two performance levels.

12.5.2.11 Hydrodynamic Force (WAD)

Hydrodynamic pressure effects acting on submerged portions of structures due to dynamic motion shall be computed using the method of Goyal and Chopra or by equivalent means.

For possible additional hydrodynamic force effects, see the Geotechnical reports described in the *Geotechnical* chapter.

12.5.2.12 Dynamic Earth Pressures (ED)

Dynamic earth pressure due to seismic motion acting on retaining structures shall be computed using the methods presented in the *Geotechnical* chapter.

12.5.2.13 Derailment Loads (DR)

A. LLRR and LLV

In the event of derailment, the damage to the bridge or aerial structure shall be minimal. Overturning or collapse of the structure shall not be allowed.

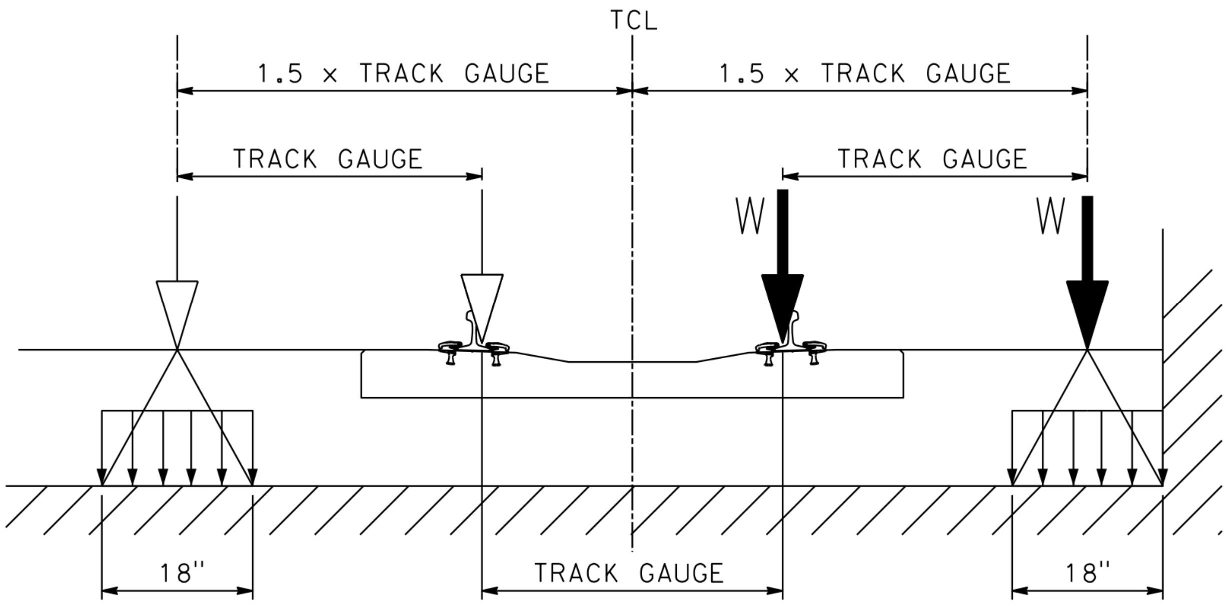
The following design situations shall be considered:

- Case I – Derailment of railway vehicles, with the derailed vehicles remaining in the track area on the bridge deck with vehicles retained by the adjacent rail or a containment wall.
- Case II – Derailment of railway vehicles, with the derailed vehicles balanced on the edge of the bridge and loading the edge of the superstructure (excluding non-structural elements such as walkways).
- Case III – Derailment of a steel wheel impacting the bridge deck between containment barriers shall be evaluated using the heaviest axle loads that potentially use the structure with a minimum of the Cooper E-50. In shared use corridors, larger axle loads shall be considered with the larger axle loads. A 100 percent impact factor shall be applied. This force is used to design the concrete deck slab. See Item B of this section.
- Case IV – Derailment of railway vehicles on a through or semi-through type aerial structure or bridge shall be designed such that the sudden rupture of one vertical or diagonal member of the truss shall not cause collapse of the structure.

For Case I, collapse of any part of the structure is not permitted. Local minor damage, however, is tolerated. The structure shall be designed for the following design loads in the Extreme Loading Combination:

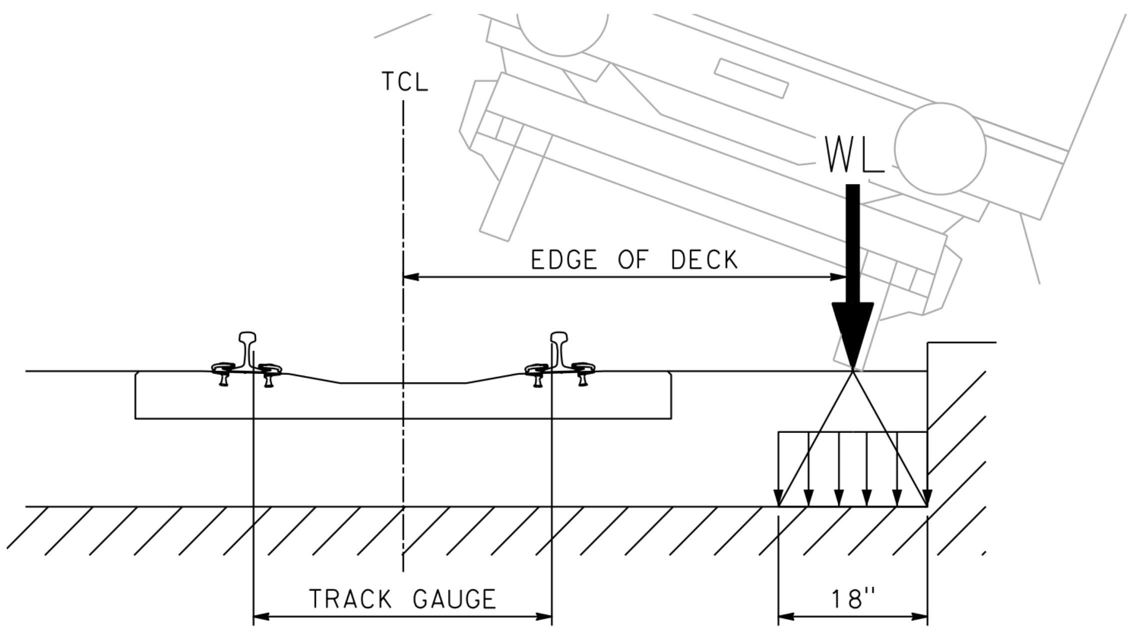
- Cooper E-50 loading, (both point loads W and uniformly distributed loading w) parallel to the track in the most unfavorable position inside an area of width 1.5 times the track gauge on either side of the TCL, or as limited by containment walls. If a short containment wall is used for containment of the train within 1.5 times the track gauge, a coincident horizontal load perpendicular to the track direction shall be used. This horizontal load shall be applied at the top of the containment wall.

Figure 12-8: Derailment Case I



For Case II, the bridge shall not overturn or collapse. For the determination of overall stability a maximum total length of 65 feet of Cooper E-50 uniform load shall be taken as a single uniformly distributed vertical line load, WL, acting on the edge of the structure under consideration. For structures with containment walls, this load shall be applied at the wall face.

Figure 12-9: Derailment Case II



Cases I and II shall be examined separately. A combination of these loads need not be considered.

For ballasted track, lateral distribution of wheel load may be applied, as shown in Figure 12-8 and Figure 12-9.

For Cases I and II, other rail traffic actions shall be neglected for the track subjected to derailment actions. When the structure under consideration carries more than 1 track, only 1 train shall be considered to have derailed, with other tracks containing a vehicle without impact if producing an unfavorable action. Multiple live load presence factors shall apply in this case.

No dynamic factor needs to be applied to the derailment loads for Case 1 and Case II. However, the loads shall be multiplied by the load factor within load combinations. A load factor of 1.0 shall apply to Case III and Case IV.

B. Track Side Containment

Containment walls shall be provided on mainline aerial structures at locations 6 feet minimum to 7 feet maximum from TCL toward the outside edge of deck. The height of the wall shall be minimum 0.67 feet above the level of the adjacent track's lower rail. The containment walls shall terminate at the back of abutment backwalls. A transverse horizontal concentrated load of 35 kips shall be applied at top of the wall at any point of contact. The load shall be distributed over a longitudinal length of 1 foot. A load factor of 1.4 shall be applied to the 35-kip load.

12.5.2.14 Collision Loads (CL)

Collision loads in Sections 12.5.2.14-A, 12.5.2.14-B, and 12.5.2.14-C apply to train impact loads (LLRR, LLV). Section 12.5.2.14-D applies to highway collision loads (LLH). For collision loads on columns or divider walls within the trackway, reference Section 12.7 - Structural Design of Surface Facilities and Buildings.

A. Collision Loads other than at Stations or Platforms

Unprotected structural members within 25 feet of the TCL shall be designed to resist train collision forces of 900 kips parallel and 400 kips perpendicular to the tracks. The loads are applied to a strip 6 feet in length and at a height centered 4 feet above grade. Forces are not applied simultaneously.

B. Collision Loads on Separation Barriers to Deter Intrusion of Derailed Freight/Passenger Trains

The height of barrier wall shall be as shown in the *Rolling Stock and Vehicle Intrusion Protection* chapter. The wall shall be constructed of reinforced concrete. The wall shall extend 15 feet beyond each end of the pier or a wall that is within 25 feet of the TCL, and shall conform to the end conditions presented in the *Rolling Stock and Vehicle Intrusion Protection* chapter.

A moving load of 400 kips transverse to the TCL applied to a strip 6 feet in length at and height centered 4 feet above ground level. A 900 kip longitudinal force shall be applied to the wall at the same elevation. Loads are not applied simultaneously.

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C. Structures in Areas beyond Track Ends

Overrunning of rail traffic beyond the end of a track (for example at a terminal station) shall be taken into account as an accidental design situation when the structure or its supports are located in the area immediately beyond the track ends.

The measures to manage the risk shall be based on the utilization of the area immediately beyond the track end and take into account any measures taken to reduce the likelihood of an overrun of rail traffic.

Members supporting structures shall not be located in the area immediately beyond the track ends.

Where structural supporting members are required to be located near to track ends, an end impact wall shall be provided within 20 feet of the track ends in addition to any buffer stop.

The design values for the static equivalent force due to impact on the end impact wall are F_{dx} = 1125 kips for passenger trains and F_{dx} = 2250 kips for freight trains or heavy engines pulling conventional passenger cars. It is recommended that these forces be applied horizontally to a 6-foot wide strip at a level of 4 feet above track level.

D. Highway Vehicle Collision Loads (LLH)

Highway collision load shall be as per AASHTO LRFD with Caltrans Amendments Article 3.6.5.

12.5.3 Miscellaneous Loads

12.5.3.1 Loads and Load Combinations for Design of the Surrounding Area of the Embedded Sleeves of Overhead Contact System Pole Foundation

The embedded sleeves as specified in the Table 12-2 for the OCS pole and down guy anchors in the outside compartment of the cable trough on both sides of structural deck shall be installed at an equal spacing not more than 30 feet in each span along the aerial structure and the offset distance from the centerline of the pier to the centerline of sleeve pattern shall be equal to 1/2 of the equal spacing. If this requirement results in a location within 6 feet of a bridge expansion joint, the location shall be adjusted to provide a spacing of no more than 30 feet to the adjacent OCS foundation anchor point. The loads, load combinations, and limit states specified in Table 12-2 shall be investigated for design of the surrounding area of the embedded sleeves at every OCS foundation to properly transfer the loads to the bridge deck.

Table 12-2: Loads for Design of Overhead Contact System Pole Foundation

Load Combination / Limit State	Location	Load Type	V1 (lbs)	V2 (lbs)	P (lbs)	M1 (lb-ft)	M2 (lb-ft)	Load Factor
Strength I	OCS pole	Dead	1,500	1,500	-22,000	31,500	31,500	1.25
		Wind	See note 4					1.40
Strength II	OCS pole	Dead	1,500	1,500	-22,000	31,500	31,500	1.25
		Slipstream Effects	See note 4					1.75
Strength III	OCS pole	Dead	1,500	1,500	-22,000	31,500	31,500	1.25
		Wind	See note 4					0.65
		Slipstream Effects	See note 4					1.35
Strength IV	Down guy anchor	Dead	14,000	1,000	14,000	1,000	14,000	1.25
Extreme I	OCS pole	Accident	14,000	1,500	-8,000	31,500	294,000	1.0
		Slipstream Effects	See note 4					1.0

Sleeve Pattern and Plate Size			
Anchor Bolt	**Bolt Circle**	**Sleeve Size**	**Plate Size**
4-2.25" Dia.	24"	2.5"	24" x 24"

Notes:

1. V_1 denotes shear force along with axis parallel to track alignment; V_2 denotes shear force along with axis perpendicular to the track alignment.
2. P denotes vertical force, with positive values for tension and negative values for compression.
3. M_1 denotes bending moment about axis parallel to the track alignment; M_2 denotes bending moment about axis perpendicular to the track alignment.
4. Wind load shall be determined according to Section 12.5.2.6; Slipstream effects load shall be determined according to Section 12.5.2.7. Wide flange shape with width of 15 inches, depth of 15 inches, and height of 27 feet shall be used in the calculations of OCS pole foundation loads transferred from OCS pole wind load and slipstream effects load.
5. Loads are assumed at the TOR.

12.5.3.2 Loads for Design of Traction Power Facility Gantry Pole Foundation

The loads specified in Table 12-3 shall be considered for design of each pole foundation of the Traction Power Facility Gantry. On aerial structures, the poles of the gantry are located outside of the cable trough on both sides of structural deck. A total of four gantry pole foundations with spacing of 10 feet, 26 feet, and 10 feet between centerlines of poles shall be assumed on each side of the designated span for the traction power facility gantry. The designer shall design the surrounding areas of the gantry pole foundations to properly transfer the loads to the bridge deck.

Table 12-3: Loads and Load Combinations for Design of Traction Power Facility Gantry Pole Foundation

Load Combination / Limit State	Load Type	V1 (lbs)	V2 (lbs)	P (lbs)	M1 (lb-ft)	M2 (lb-ft)	Load Factor
Strength I	Dead	--	4,500	-11,000	180,000	--	1.25
	Wind	See note 4					1.40
Strength II	Dead	--	4,500	-11,000	180,000	--	1.25
	Slipstream Effects	See note 4					1.75
Strength III	Dead	--	4,500	-11,000	180,000	--	1.25
	Wind	See note 4					0.65
	Slipstream Effects	See note 4					1.35
Strength IV	Dead	--	4,500	-11,000	180,000	--	1.25
	Slipstream Effects	See note 4					0.5
	OBE	See note 5					1.0
Extreme I	Dead	--	4,500	-11,000	180,000	--	1.0
	MCE	See note 5					1.0

Notes:

1. V_1 denotes shear force along with axis parallel to track alignment; V_2 denotes shear force along with axis perpendicular to the track alignment.
2. P denotes vertical force, with positive values for tension and negative values for compression.
3. M_1 denotes bending moment about axis parallel to the track alignment; M_2 denotes bending moment about axis perpendicular to the track alignment.
4. Wind load shall be determined according to Section 12.5.2.6; Slipstream effects load shall be determined according to Section 12.5.2.7. Wide flange shape W24x117 with height of 40 feet, with 100% and 300% wind load area increases in along track and transverse directions respectively to account for the cross beams and attachments, shall be used in the calculations of gantry pole foundation loads transferred from gantry wind load and slipstream effects load.
5. Operating Basis Earthquake (OBE) and Maximum Considered Earthquake (MCE) shall be investigated according to the *Seismic* chapter. The pole shall be considered as cantilever, with weight of 11,000 lbs and the center of mass at 27 feet above the TOR.
6. Loads are assumed at the TOR.

12.5.3.3 Construction Loads and Temporary Structures

A. Temporary Structure Classification

Temporary structures are divided into the following classifications:

- Type A – Temporary structures or structures under temporary conditions which carry or will carry HSTs and/or pass over routes carrying HSTs. Subsequent articles herein apply to Type A structures.
- Type B: – Temporary structures or structures under temporary conditions which do not carry HSTs and do not pass over routes carrying HSTs. These structures shall be designed in accordance with the requirements of the owning/operating agency (e.g., AASHTO LRFD with Caltrans Amendments and CMTD). Structures such as haul bridges used temporarily shall be designed in accordance with CMTD 15-14.

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For earth retaining structures and underground structures, see requirements in the *Geotechnical* chapter.

B. Construction Load Combinations

Type A temporary structures or structures under temporary conditions shall be designed to adequately resist conditions at all stages of construction, including applicable construction loads. Construction load combinations, in addition to requirements of Table 12-4, shall include the following:

- Applicable strength load combinations: Dead load factors shall not be taken less than 1.25, with construction dead loads taken as permanent loads. Construction transient live load factors shall not be taken less than 1.5. Wind load factors may be reduced by 20 percent.
- Service 1, as applicable, see AASHTO LRFD Article 3.4.2.
- Seismic requirements for temporary structures are presented in the *Seismic* chapter.

In the absence of specific criteria, a construction live load of 20 psf shall be assumed on the bridge deck.

C. Segmental Construction and Specialized Equipment

Construction load combinations per AASHTO LRFD with Caltrans Amendments Article 5.14.2 "Segmental Construction" shall be considered. The temporary seismic load event as described in the *Seismic* chapter shall be added to the construction load combination at Strength 5 limit state; however a 1.25 load factor shall be used for dead and live loads. The temporary seismic event need not be combined with the dynamic construction load impact due to segment drop or equipment impact. For balanced cantilever construction methods an additional 2 percent of dead load shall be applied to reflect eccentric conditions at the time of a potential seismic event.

12.5.3.4 Track-Structure Interaction Forces

Effects of continuous welded rail (CWR) interaction with the structure through its attachment shall be considered. Design guidance and requirements for this interaction is provided in Section 12.6 - Track-Structure Interaction.

12.5.3.5 Blast Loading

Blast loadings and measures are not specified at this time. See Section 12.13 for general requirements. Refer to AASHTO LRFD with Caltrans Amendments Article 3.15 for aerial structure requirements.

12.5.4 Load Factors and Load Modifiers

Regardless of the type of analysis used, the Eq. 12.5.4-1 shall be satisfied for specified factored force effect and load combinations for each limit state unless otherwise specified in the section:

$$\Sigma\ \eta_i\ \gamma_i\ Q_i \leq \Phi R_n = R_r \quad \text{(Eq. 12.5.4-1)}$$

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Where:

γ_i = load factor applied to force effects (see Tables 12-4, 12-6 and 12-7)

Φ = resistance factor applied to nominal resistance (see AASHTO LRFD with Caltrans Amendments Article 1.3.2.1)

η_i = load modifier relating to ductility, redundancy and importance (see AASHTO LRFD with Caltrans Amendments Article 1.3.2.1)

Q_i = force effect

Rn = nominal resistance

Rr = factored resistance, ΦRn

For loads in which a maximum value of "η_i" produces an unfavorable action, the value of "η_i" shall be equal to 1.05 to account for the design life of the facility. The load modifier is applicable to Strength Limit Load Combinations only.

12.5.4.1 Design Load Combinations

The load combinations to be used for structures carrying HSTs are presented in Table 12-4. The description of the load combinations are as follows:

- "Strength 1" is the basic load combination for normal use.
- "Strength 2" is the load combination for the structure exposed to wind.
- "Strength 3" is the load combination for very high dead load to live load force effect ratios.
- "Strength 4" is the load combination for normal use when exposed to wind.
- "Strength 5" is the load combination for normal use when designing columns for OBE.
- "Extreme 1" is the load combination for derailment.
- "Extreme 2" is the load combination for collision.
- "Extreme 3" is the load combination for extreme seismic events: MCE.
- "Service 1" is the basic service load combination for normal use with wind.
- "Service 2" is the service load combination intended to control yielding of steel structures and slip of slip-critical connections due to train load.
- "Service 3" is the service load combination relating to tension in prestressed concrete superstructures with the objective of crack control and principal tension in the webs of segmental concrete girders.
- "Buoyancy at Dewatering Shutoff" is a service load for evaluation of uplift with a minimum weight structure.
- "Fatigue" is the fatigue and fracture load combination relating to repetitive vertical train loading.

Note that for each load combination, physically achievable subsets (i.e., omitting loads by setting load factor $\gamma_i = 0$) which may govern design shall be considered.

Note that other load cases for train and track structure interaction are contained within Section 12.6 on Track-Structure Interaction.

Table 12-4: Load Combinations for Design of Structures

Load Combinations and Load Factors, γ_i Load Combination/ Limit State	DC DW DD EV EH ES EL PS CR SH	LLP LLV + I LLRR + I LLH + I LLS LF NE CF SS	WA FR	WS	WL	TU	TG	SE	DR	CL	OBE	MCE WAD ED
Strength 1	γ_P	1.75	1.00	--	--	0.50/ 1.20	--	γ_{SE}	--	--	--	--
Strength 2	γ_P	--	1.00	1.40	--	0.50/ 1.20	--	γ_{SE}	--	--	--	--
Strength 3	γ_P	--	1.00	--	--	0.50/ 1.20	--	--	--	--	--	--
Strength 4	γ_P	1.35	1.00	0.65	1.00	0.50/ 1.20	--	γ_{SE}	--	--	--	--
Strength 5	γ_P	γ_{EQ}	1.00	--	--	--	--	--	--	--	1.0	--
Extreme 1	1.00	1.00	1.00	--	--	--	--	--	1.40	--	--	--
Extreme 2	1.00	0.50	1.00	--	--	--	--	--	--	1.00	--	--
Extreme 3	1.00	γ_{EQ}	1.00	--	--	--	--	--	--	--	--	1.00
Service 1	1.00	1.00	1.00	0.45	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}	--	--	--	--
Service 2	1.00	1.30	1.00	--	--	1.00/ 1.20	--	--	--	--	--	--
Service 3	1.00	1.00	1.00	--	--	1.00/ 1.20	γ_{TG}	γ_{SE}	--	--	--	--
Buoyancy @ Dewatering Shutoff	0.80	--	1.10	0.45	--	--	--	--	--	--	--	--
Fatigue	--	1.00	--	--		--	--	--	--	--	--	--

Notes:

1. Additional load combinations are found in Section 12.6 – Track-Structure Interaction
2. Additional loads and load combinations for cut-and-cover construction are found in the Technical Manual for Design and Construction of Road Tunnels – Civil Elements; FHWA-NHI-09-010, March, 2009, Chapter 5
3. γ_{TG} is equal to 1.0 when live load is not considered and 0.50 when live load is considered.
4. γ_{EQ} is equal to 0.0 for MCE. γ_{EQ} is equal to 0.50 for OBE, for a two track system, one train is used. For other track configurations, refer to the *Seismic* chapter.
5. γ_{SE} should equal 1.0, in absence of better criteria. For specific areas where settlement values are uncertain, or if otherwise justified, a larger value should apply.

6. γ_{TU} is equal to the larger value for deformations, and the lesser value for force effects.
7. Derailment load factor taken greater than unity to account for absence of dynamic impact. Refer to Section 12.5.2.13- A.
8. WS load factors for Service I and Strength 4 are larger than the AASHTO LRFD with Caltrans Amendments to account for a higher wind speed under train operations. Operation of trains is assumed to cease at a wind speed of 67 mph.

Table 12-5: Loading Definitions Used in Table 12-4

Permanent Loads	
DC	dead load of structural components and permanent attachments
DW	dead load of non-structural and non-permanent attachment
DD	downdrag force
EV	vertical earth pressure
EH	lateral static earth pressure
ES	surcharge loads
SE	earth settlement effects
EL	locked-in construction forces
PS	secondary forces from prestressing
CR	creep effects
SH	shrinkage effects
WA	water loads & stream pressure
Transient Loads	
LLP	floor, roof, and pedestrian live loads
LLV	high-speed train live loads
LLRR	maintenance and construction train live loads
LLH	highway live loads
LLS	live load surcharge
I	vertical impact effect
LF	traction or braking forces
NE	nosing and hunting effects
CF	centrifugal force
DR	derailment loads
CL	collision loads
WS	wind load on structure
WL	wind load on live load
SS	slipstream effects
TU	uniform temperature effects
TG	gradient temperature effects
FR	frictional force
MCE	Maximum Considered Earthquake
OBE	Operating Basis Earthquake
WAD	hydrodynamic force
ED	dynamic earth pressures

Table 12-6: Load Factors for Permanent Loads, γ_P

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		γ_p Load Factor Maximum	γ_p Load Factor Minimum
DC: Components and Attachments		1.25	0.90
DC: Strength 3 only		1.50	0.90
DD: 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
DW: Non-structural dead load and non-permanent attachments		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• AEP for Anchored Walls		1.35	N/A
EL: locked-in construction forces		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structures		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
ES: Surcharge Loads		1.50	0.75

Table 12-7: Load Factors for Permanent Loads due to Superimposed Deformations, γ_P

Bridge Component	PS	CR, SH
Superstructures - Segmental Concrete Substructures supporting Segmental Superstructures (See AASHTO LRFD with Caltrans Amendments Articles 3.12.4 and 3.12.5)	1.00	see γ_p for DC, Table 12-4
Concrete Superstructures – non-segmental	1.00	1.00
Substructures supporting non-segmental superstructures		
• Using I_{gross}	0.50	0.50
• Using $I_{effective}$	1.00	1.00
Steel Substructures	1.00	1.00

12.5.4.2 Resistance Factors

For resistance factors Φ, refer to AASHTO LRFD with California Amendments.

12.6 Track-Structure Interaction

Bridges and aerial structures that support high-speed trains are subject to structural frequency limits, track serviceability limits, rail-structure interaction limits, dynamic structural analysis limits, and dynamic vehicle track-structure interaction analysis limits.

These requirements are concerned with limiting bridge and aerial structure deformations and accelerations, since the structure response can be dynamically magnified under high-speed moving trains. Excessive deformations and accelerations can lead to unacceptable changes in vertical and horizontal track geometry, excessive rail stress, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

Track-structure interaction criteria shall apply to any aerial structure whose primary function is to support high-speed train (HST) tracks and therefore critical to track performance. These aerial structures include, but are not limited to: bridges, grade separations, stream crossings, canal crossings, culverts with less than 6 feet of cover, viaducts, and aerial stations supporting HST tracks. For a given aerial structure, all structural discontinuities capable of relative movement shall be considered a structural expansion joint and subject to the applicable criteria.

For criteria related to embankments, abutments, retaining walls, and soil subgrades critical for ensuring track performance, see the *Geotechnical* chapter. For criteria related to track performance in tunnels, see *Tunnels* chapter. Track support discontinuities at transitions between aerial structures and geotechnical elements (including tunnels) shall be considered structural expansion joints and subject to the following applicable criteria.

The following criteria is developed assuming uniform longitudinal rail restraint for structures with maximum structural thermal units (i.e., the maximum distance from a fixed point of thermal expansion to an adjacent fixed point of thermal expansion on a structure) limited to 330 feet. Rail expansion joints are not permitted without an approved design variance.

Table 12-8 summarizes the analysis requirements, including model type, train model/speed, result, and relevant section.

Table 12-8: Analysis Goals

Analysis Goal	Model Type	Train model	Train speed	Result	Section(s)
Frequency Analysis	Dynamic	--	--	Frequency Evaluation	12.6.3.1 to 12.6.3.3
Track Serviceability Analysis	Static, For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation Limits	12.6.4.1 to 12.6.4.10
Rail-Structure Interaction Analysis	Static (linear or non-linear), For OBE: Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation And Rail Stress Limits	12.6.5.1 to 12.6.5.6
Dynamic Structural Analysis	Dynamic	Single Tracks of High-Speed Passage	90 mph to 1.2 Line Speed (or 250 mph whichever is less)	Dynamic Impact Factor, Vertical Deck Acceleration	12.6.6.1 to 12.6.6.4
Dynamic Vehicle-Structure Interaction Analysis	Dynamic (Structure & Trainset)	Single Track of High-Speed Passage (with Vehicle Suspension)	90 mph to 1.2 Line Speed (or 250 mph whichever is less)	Dynamic Track Safety and Passenger Comfort Limits	12.6.7.1 to 12.6.7.3

12.6.1 Design Requirements

Frequency analysis, track serviceability analysis, rail-structure interaction analysis, and dynamic structural analysis, shall apply for all structures (Standard, Non-Standard, and Complex structures per *Seismic* chapter).

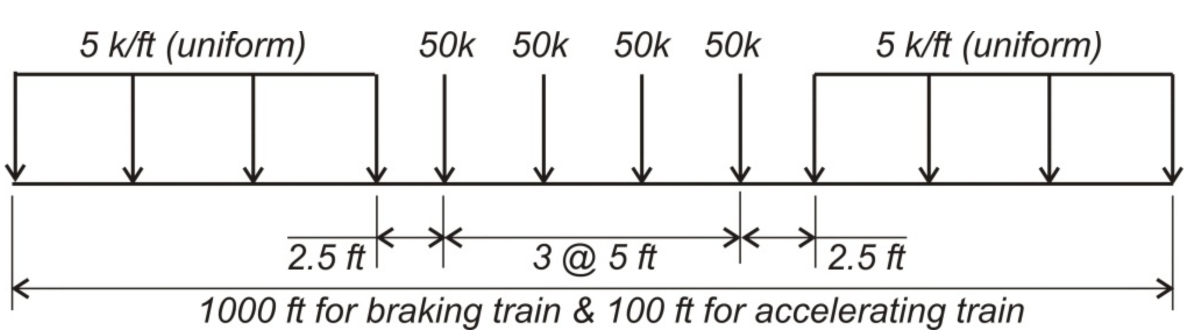
12.6.2 Design Parameters

The following defines loading to be used for track serviceability and rail-structure interaction analysis.

12.6.2.1 Modified Cooper E-50 Loading (LLRM)

Modified Cooper E-50 loading (LLRM) in Figure 12-10 shall be used for track serviceability analysis, and rail-structure interaction analysis. LLRM loading is on a per track basis.

Figure 12-10: LLRM Loading



Nosing effects do not apply to LLRM loading.

12.6.2.2 Vertical Impact Effect (I)

The vertical impact effect (I) used with Modified Cooper E-50 loading (LLRM) shall be per Section 12.5.

Dynamic vertical impact effects (I_{LLV}) caused by high-speed trainsets (LLV) shall be per Section 12.6.6.3.

12.6.3 Frequency Analysis

Frequency limits are placed on the fundamental mode shapes of bridges and aerial structures, in order to ensure well-proportioned structures and minimize resonancy effects.

Modeling requirements are given in Section 12.6.8.

12.6.3.1 Recommended Range of Vertical Frequency of Span

The recommended vertical frequency range is known to favorably resist high-speed train resonance actions. It is recommended that structures be proportioned to fall within this range. Where a structure falls outside the recommended vertical frequency range, additional analysis shall be required per Section 12.6.1.

Vertical frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For vertical frequency analysis, two conditions shall be investigated:

- Condition #1 – a lower bound estimate of stiffness and upper bound estimate of mass.
- Condition #2 – an upper bound estimate of stiffness and lower bound estimate of mass.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 12.6.8.

The recommended range for the first natural frequency of vertical deflection, η_{vert} [Hz], primarily due to bending of the span is the following:

$$\eta_{lower} \leq \eta_{vert} \leq \eta_{upper}$$

Where:

$\eta_{lower} = 262.5/L$ for 13 ft $\leq L \leq$ 66 ft, or

$\eta_{lower} = 47.645L^{-0.592}$ for 66 ft $\leq L \leq$ 330 ft, and

$\eta_{upper} = 230.46L^{-0.748}$, for 13 ft $\leq L \leq$ 330 ft, and

L = effective length of span (feet)

For simple spans, L shall be the span length.

For continuous spans, L shall be the following:

$$L = k\left(L_{\text{average}}\right)$$

Where:

$$L_{\text{average}} = \frac{(L_1 + L_2 + ... + L_n)}{n}$$ = the average span length,

$n =$ the number of spans

$$k = \left(1 + \frac{n}{10}\right) \leq 1.5$$

For portal frames and closed frame bridges, L shall be the following:

- Single span: consider as three continuous spans, with the first and third span being the vertical length of the columns, and the second span the girder length.
- Multiple spans: consider as multiple spans, with the first and last span as the vertical length of the end columns, and the interior spans the girder lengths.

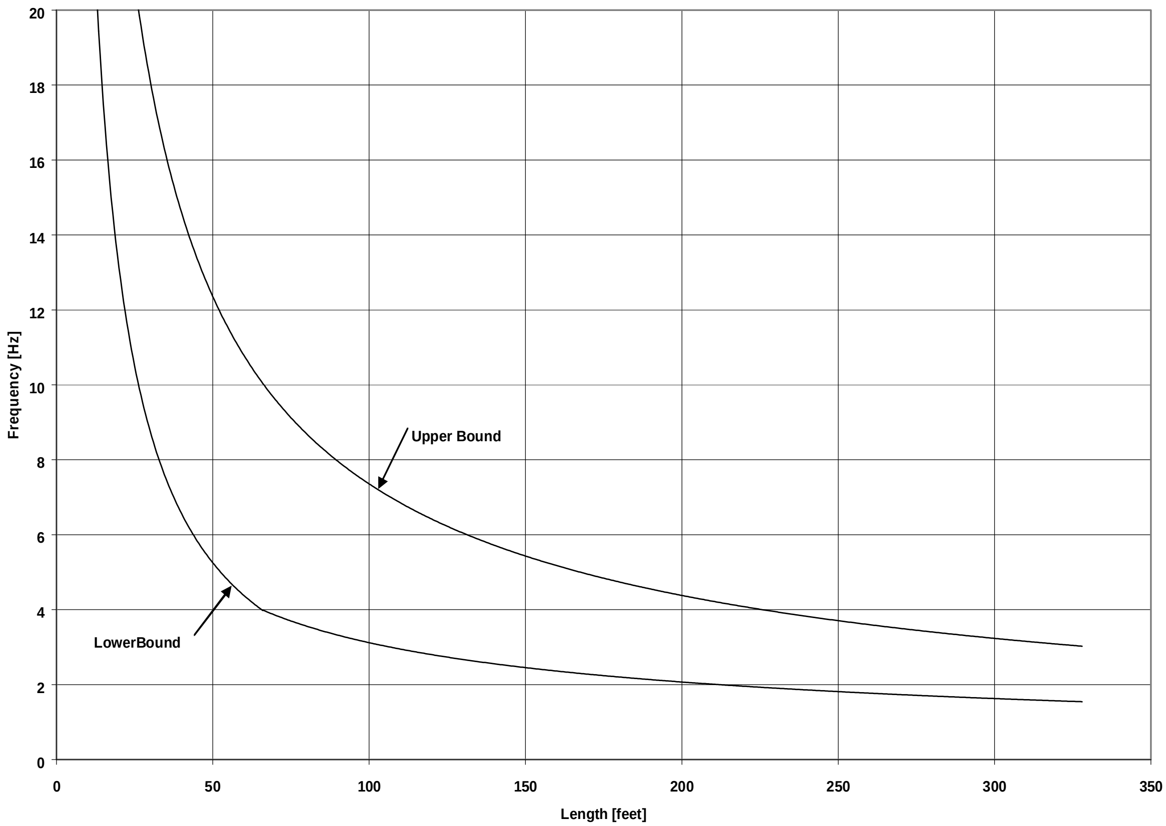
For spans with end diaphragms at abutments (fixed supports at abutments):

- Single span, fixed at one abutment: consider as two continuous spans, with the first span equal to 0.05 times the girder length, and the second span the girder length.
- Single span, fixed at both abutments: consider as three continuous spans, with the first and the third span equal to 0.05 times the girder length, and the second span the girder length.
- Multiple spans, fixed at one abutment: consider as multiple spans, with the first span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.
- Multiple spans, fixed at both abutments: consider as multiple spans, with the first and last span equal to 0.05 times the adjacent girder length, and the interior spans the girder lengths.

For single arch, archrib, or stiffened girders of bowstrings, L shall be the half span.

See Figure 12-11 for the recommended range of vertical frequency.

Figure 12-11: Recommended Range of Vertical Frequency



12.6.3.2 Allowable Torsional Frequency of Span

Limiting the allowable torsional frequency favorably resists high-speed train actions.

Torsional frequency analysis shall consider the flexibility of superstructure, bearings, shear keys, columns, and foundations.

For torsional frequency analysis, two conditions shall be investigated, consistent with vertical frequency analysis:

- Condition #1 – a lower bound estimate of stiffness and upper bound estimate of mass
- Condition #2 – an upper bound estimate of stiffness and lower bound estimate of mass

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 12.6.8.

The first torsional frequency, $\eta_{torsion}$, of the span shall be greater than 1.2 times the first natural frequency of vertical deflection, η_{vert}.

12.6.3.3 Allowable Transverse Frequency of Span

Limiting the allowable transverse frequency favorably resists high-speed train actions.

Transverse frequency analysis shall consider the flexibility of superstructure only, excluding the flexibility of bearings, columns, and foundations, assuming the supports at the ends of the span are rigid.

For transverse frequency analysis, a lower bound estimate of stiffness and upper bound estimate of mass shall be used, see Section 12.6.8.

The first natural frequency of transverse deflection, η_{trans}, of the span shall not be less than 1.2 Hz.

12.6.4 Track Serviceability Analysis

Track serviceability analysis, using modified Cooper E-50 loading, provides limits to allowable structural deformations. These track serviceability limits are developed for structures supporting continuous welded rail without rail expansion joints.

Deformation limits are developed for limit states based on maintenance, passenger comfort, and track safety requirements.

For track serviceability analysis, the flexibility of superstructure and substructure (i.e: bearings, shear keys, columns, and foundations) shall be considered.

To avoid underestimating deformations, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Modeling requirements are given in Section 12.6.8.

12.6.4.1 Track Serviceability Load Cases

Track serviceability loads cases shall include the following:

- Group 1a: $(LLRM + I)_1$
- Group 1b: $(LLRM + I)_2 + CF_2 + WA$
- Group 1c: $(LLRM + I)_m + CF_m + WA$
- Group 2: $(LLRM + I)_1 + CF_1 + WA + WS + WL_1$
- Group 3: $(LLRM + I)_1 + CF_1 + OBE$

Where:

$(LLRM + I)_1$ = one track of Modified Cooper E-50 (LLRM) plus impact

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$(LLRM + I)_2$ = two tracks of Modified Cooper E-50 (LLRM) plus impact (Section 12.5.2.1-D)

$(LLRM + I)_m$ = multiple tracks of Modified Cooper E-50 (LLRM) plus impact

I = vertical impact factor from LLRR (Section 12.5.2.2)

CF_1 = centrifugal force (one track) (Section 12.5.2.3)

CF_2 = centrifugal force (two tracks) (Section 12.5.2.3)

CF_m = centrifugal force (multiple tracks)

WA = water loads (stream flow) (Section 12.5.2.10)

WS & WL_1 = wind on structure and wind on one 1000' LLRM train (Section 12.5.2.6)

OBE = Operability Basis Earthquake per *Seismic* chapter

Note that Group 1c is used for Section 12.6.4.4 only.

Static analysis and linear superposition of results is allowed for Groups 1b, 1c, and 2.

For determining OBE demands in Group 3, equivalent static analysis, dynamic response spectum, or time history (linear or non-linear) analysis shall be used, as per the *Seismic* chapter and the approved Seismic Design and Analysis Plan. See the *Seismic* chapter for additional OBE modeling requirements.

For track serviceability analysis, non-linear track-structure interaction modeling (see Section 12.6.8.5) is not required, but may be used. For Group 3, superposition of static (i.e., $(LLRM + I)_1 + CF_1$) and either static or dynamic OBE shall be allowed.

12.6.4.2 Vertical Deflection Limits: Group 1a

Vertical deflection limits for Group 1a are to address maintenance, passenger comfort, and track safety issues.

For Group 1a, the maximum static vertical deck deflection (max Δ_{1a}), in the most unfavorable position, shall not exceed the limits given in Table 12-9.

Table 12-9: Vertical Deflection Limits: Group 1a

Limit	Span Length (feet)				
	L ≤ 125	L=175	L=225	L=275	L≥330
max Δ_{1a}	L/3500	L/3180	L/2870	L/2550	L/2200

For span lengths not explicitly referenced in Table 12-9, linear interpolation shall be used.

12.6.4.3 Vertical Deflection Limits: Group 1b

Vertical deflection limits for Group 1b are to address maintenance, passenger comfort, and track safety issues.

For Group 1b, the maximum static vertical deck deflection (max Δ_{1b}), in the most unfavorable position, shall not exceed the limits given in Table 12-10.

Table 12-10: Vertical Deflection Limits: Group 1b

Limit	Span Length (feet)				
	L ≤ 125	L=175	L=225	L=275	L≥330
max Δ_{1b}	L/2400	L/2090	L/1770	L/1450	L/1100

For span lengths not explicitly referenced in Table 12-10, linear interpolation shall be used.

12.6.4.4 Vertical Deflection Limits: Group 1c

Vertical deflection limits for Group 1c are to provide practical guidance for structures containing three or more tracks operating at speeds less than 90 mph. This guidance is consistent with established European codes.

For Group 1c, where the structures support three or more tracks, the loading shall be applied per Section 12.5.2.1-D.

The tracks selected for loading shall be those tracks which will produce the most critical design condition on the member under consideration.

For Group 1c, where structures support three or more tracks, the maximum static vertical deck deflection (max Δ_{1c}), in the most unfavorable position, shall not exceed L/600 for all span lengths.

In the event that structures support 3 or more tracks, and 3 or more trains can be anticipated to be on the same structure at speeds greater than 90 mph, limits defined for Group 1b shall apply. For these structures, loadings representative of service conditions must be developed on a case-by-case basis.

12.6.4.5 Transverse Deflection Limits

Transverse deflection limits are to address maintenance, passenger comfort, and track safety issues.

The transverse deflection within the span (Δ_{trans}), shown in Figure 12-12, shall not exceed the limits given in Table 12-10.

Figure 12-12: Transverse Span Deformation Limits

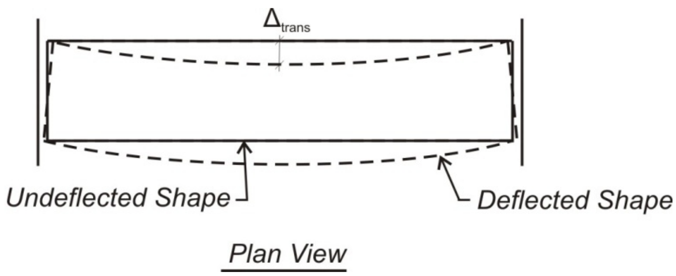


Table 12-11: Transverse Deflection Limits

Group	Δ_{trans} (feet)
1a	$L^2/(864,800)$
1b	$L^2/(447,200)$
2	$L^2/(276,800)$
3	$L^2/(276,800)$

12.6.4.6 Rotation about Transverse Axis Limits

Rotation about transverse axis limits are to control excessive rail axial and bending stress, provide traffic safety (i.e., guard against wheel unloading due to abrupt angular changes in track geometry), and provide passenger comfort.

Due to rotation about the transverse axis, imposed axial rail displacement is a linear function of the distance between the rail centroid and top of the bridge bearings. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about transverse axis at deck ends (θ_t), shown in Figure 12-13, shall be defined by the following equations:

$\theta_t = \theta$, for abutment condition

$\theta_t = \theta_1 + \theta_2$, between consecutive decks

The maximum relative axial displacement at the level of the rail (δ_t) due to rotation about transverse axis, shown in Figure 12-13, shall also be defined by the following equations:

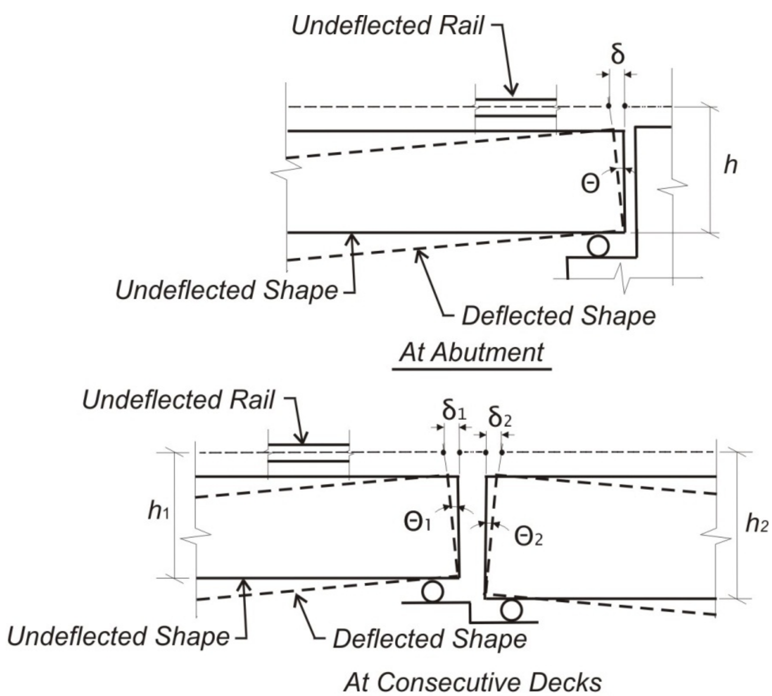
$\delta_t = \theta h$, for abutment condition

$\delta_t = \delta_1 + \delta_2 = \theta_1 h_1 + \theta_2 h_2$, between consecutive decks.

Where:

θ_t (radians)	= total rotation about transverse axis, see Table 12-12
δ_t (inches)	= total relative displacement at the level of the rail, see Table 12-12
θ (radians)	= rotation of the bridge bearing at abutment
θ_1 (radians)	= rotation of the first bridge bearing
θ_2 (radians)	= rotation of the second bridge bearing
h (inches)	= the distance between the rail centroid and the bridge bearing at abutment
h_1 (inches)	= the distance between the rail centroid and the top of the first bridge bearing
h_2 (inches)	= the distance between the rail centroid and the top of the second bridge bearing

Figure 12-13: Rotation about Transverse Axis at Deck Ends



The total rotation about transverse axis (θ_t) and the total relative displacement at the level of the rail (δ_t) shall not exceed the limits given in Table 12-12.

Table 12-12: Rotation about Transverse Axis and Relative Displacement at the Level of the Rail Limits

Group	θ_t (radians)	δ_t (inches)
1a	0.0012	0.33
1b	0.0017	0.33
2	0.0026	0.67
3	0.0026	0.67

12.6.4.7 Rotation about Vertical Axis Limits

Rotation about vertical axis limits are to control excessive rail axial and bending stress, provide track safety, and provide passenger comfort by limiting changes in horizontal track geometry at bridge deck ends.

Due to rotation about the vertical axis, imposed axial rail displacement is a linear function of the distance between the centerline of span and the outermost rail. This imposed axial displacement causes rail stress. Rail stress limits may control over passenger comfort and track safety limits.

The maximum total rotation about vertical axis at deck ends (θ_v), shown in Figure 12-14 shall be defined by the following equations:

$\theta_v = \theta$, for abutment condition

$\theta_v = \theta_A + \theta_B$, between consecutive decks

The maximum axial displacement at the outermost rail (δ_v) due to rotation about vertical axis, shown in Figure 12-15, shall be defined by the following equations:

$\delta_v = \theta w$, for abutment condition

$\delta_v = \delta_A + \delta_B = \theta_A w_A + \theta_B w_B$, between consecutive decks.

Where:

θ_v (radians): total rotation about vertical axis, see Table 12-13

δ_v (inches): total relative displacement at the outermost rail, see Table 12-13

θ (radians): rotation of the bridge at abutment

θ_A (radians): rotation of the first span

θ_B (radians): rotation of the second span w (inches): the distance between the centerline span and outermost rail centroid at abutment

w_A (inches): the distance between the centerline span and outermost rail centroid of first span

w_B (inches): the distance between the centerline span and outermost rail centroid of second span

Figure 12-14: Rotation about Vertical Axis at Deck Ends – Global View

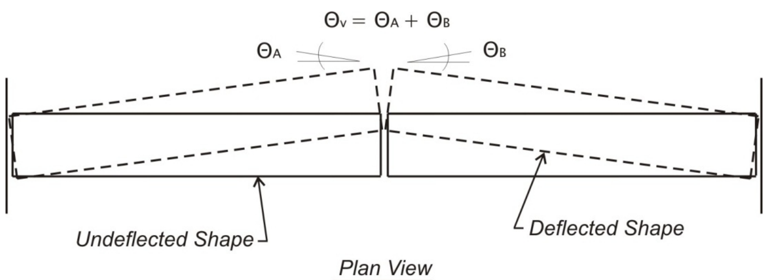
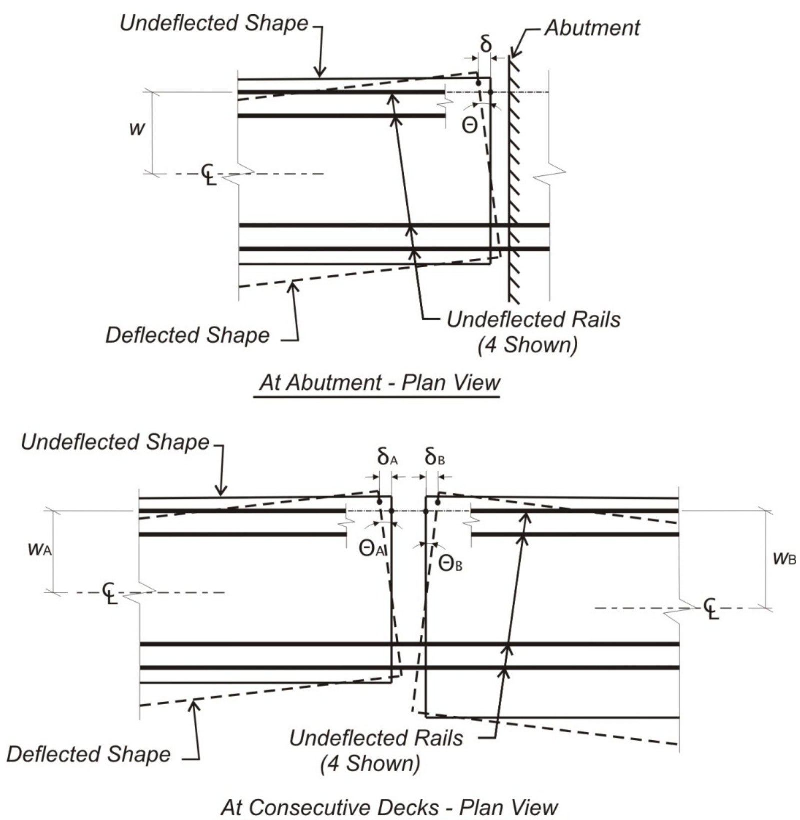


Figure 12-15: Rotation about Vertical Axis at Deck Ends – Local View



The total rotation about vertical axis (θ_v) and the total relative displacement at the outermost rail (δ_v) shall not exceed the limits given in Table 12-13.

Table 12-13: Rotation about Vertical Axis and Relative Displacement at Outermost Rail Limits

Group	θ_v (radians)	δ_v (inches)
1a	0.0007	0.33
1b	0.0010	0.33
2	0.0021	0.67
3	0.0021	0.67

12.6.4.8 Relative Vertical Displacement at Expansion Joints – Track Serviceability

Relative vertical displacements (RVD) at structural expansion joints, δ_V^{EXP}, are limited in order to ensure track safety subject to deck end rotation and vertical bearing deformation. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

The flexibility of the superstructure and substructure (i.e.: bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

The relative vertical displacement at expansion joints (δ_V^{EXP}), shown in Figure 12-16, shall not exceed the limits given Table 12-14.

See Section 12.6.5.3 for RVD limits used to prevent excessive rail stress.

Figure 12-16: Relative Vertical Displacement at Expansion Joints – Track Serviceability

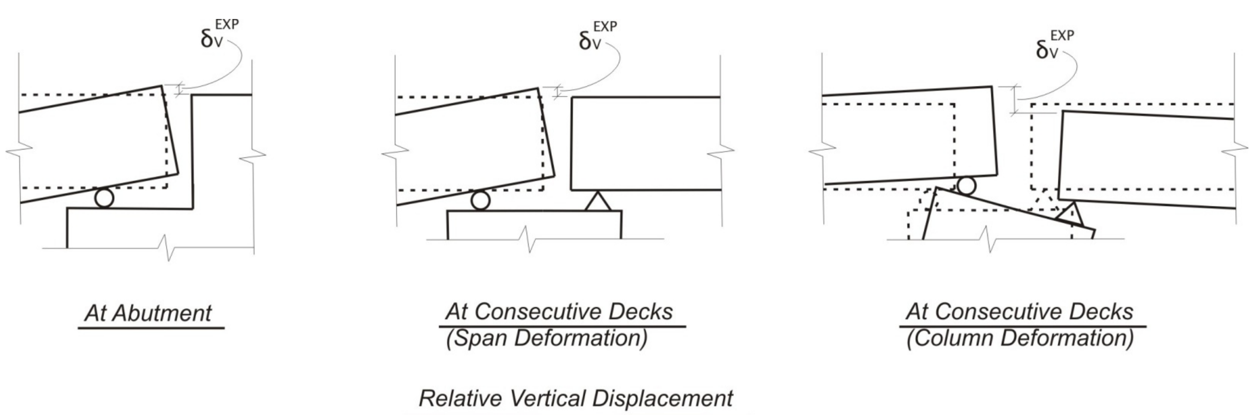


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Table 12-14: Relative Vertical Displacement at Expansion Joints Limits – Track Serviceability

Group	δ_V^{EXP} (inch)
1a	0.25
1b	0.25
2	-
3	-

12.6.4.9 Relative Transverse Displacement at Expansion Joints – Track Serviceability

Relative transverse displacements (RTD) at structural expansion joints, δ_T^{EXP}, are limited in order to ensure track safety subject to shear key and lateral bearing deformation. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

The relative transverse displacement at expansion joints (δ_T^{EXP}), shown in Figure 12-17, shall not exceed the limits given in Table 12-15.

See Section 12.6.5.4 for relative transverse deflection limits used to prevent excessive rail stress.

Figure 12-17: Relative Transverse Displacement at Expansion Joints – Track Serviceability

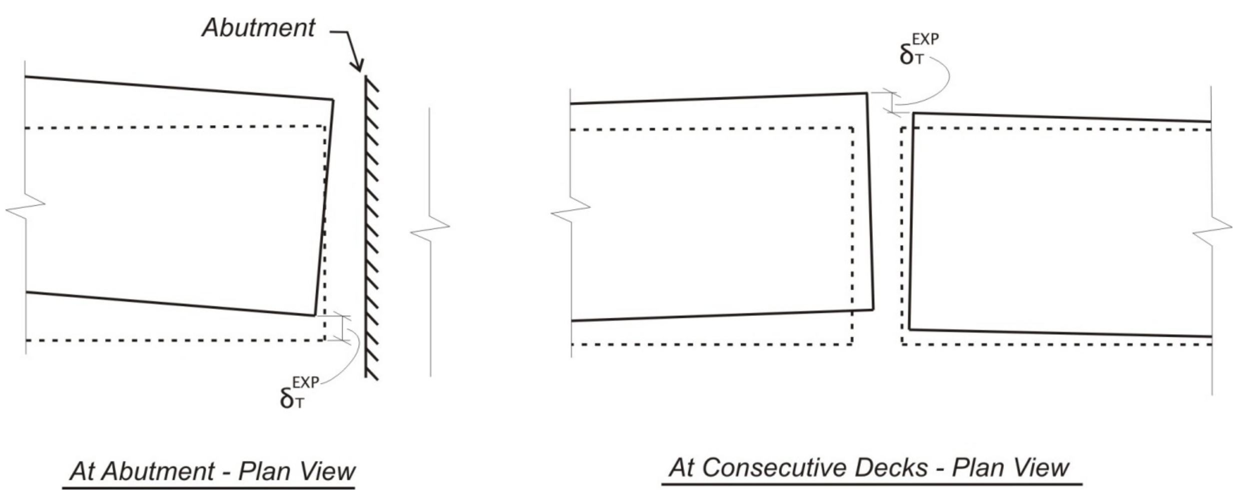


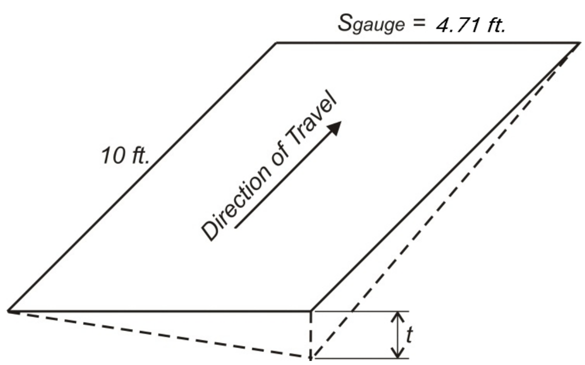
Table 12-15: Relative Transverse Displacement at Expansion Joints Limits – Track Serviceability

Group	δ_T^{EXP} (inch)
1a	0.08
1b	0.08
2	-
3	-

12.6.4.10 Deck Twist Limits

The deck twist, t, is defined as the relative vertical deck displacment of a given bogie contact point from a plane defined by the remaining three bogie contact points on a track gauge of 4.71 feet over a bogie length of 10 feet, see Figure 12-18. Deck twist limits ensure that the four wheel contact points of a bogie are not too far from a plane.

Figure 12-18: Deck Twist Diagram



Maximum deck twist (t_{max}) below tracks shall not exceed the limits given in Table 12-16.

Table 12-16: Deck Twist Limits

Group	t_{max} (inches / 10 feet)
1a	0.06
1b	0.06
2	0.17
3	0.17

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12.6.5 Rail-Structure Interaction Analysis

Rail-structure interaction analysis, using modified Cooper E-50 loading, shall be used to limit relative longitudinal, vertical, and transverse displacements at structural expansion joints, and axial rail stress. Rail-structure interaction analysis is required to limit rail stress and therefore minimize the probability of derailment due to rail fracture, Deformation limits and rail stress limits were developed considering the accumulation of displacement demands and rail bending stresses under the controlling load combinations.

For rail-structure interaction analysis, the flexibility of superstructure and substructure (i.e.: bearings, shear keys, columns, and foundations) shall be considered.

In order to avoid underestimating deformations and rail stress, a lower bound estimate of stiffness and an upper bound estimate of mass shall be used.

Details of rail-structure modeling requirements are given in Section 12.6.8.5.

12.6.5.1 Rail-Structure Interaction Load Cases

Rail-structure interaction load cases include the following:

- Group 4: $(LLRM + I)_2 + LF_2 \pm T_D$
- Group 5: $(LLRM + I)_1 + LF_1 \pm 0.5T_D + OBE$

Where:

$(LLRM + I)_1$ = single track of Modified Cooper E-50 (LLRM) plus impact

$(LLRM + I)_2$ = two tracks of Modified Cooper E-50 (LLRM) plus impact

I = vertical impact factor from LLRR

LF_1 = braking forces (where one track, apply braking) for LLV loading

LF_2 = braking and acceleration forces (apply braking to one track, acceleration to the other track) for LLV loading

T_D = temperature differential of ±40°F between rails and deck, applied to the superstructure.

OBE = Operability Basis Earthquake per *Seismic* chapter

Groups 4 and 5 are to provide relative longitudinal, vertical, and transverse displacement limits at expansion joints, and design for uplift at direct fixation rail. Groups 4 and 5 are also used to limit rail stresses, accounting for thermal effects (i.e. $\pm T_D$).

Modeling of non-linear track-structure interaction effects, as given in Section 12.6.8.5, is required to give realistic demands. Experience has shown that linear modeling of track-structure interaction is overly conservative.

For Group 5, non-linear time-history OBE analysis (i.e., non-linear rail-structure interaction) shall be used for design. $(LLRM + I)_1 + LF_1$ may be idealized as a set of stationary load vectors placed upon the structure in the most unfavorable position. See the *Seismic* chapter for additional OBE modeling requirements.

12.6.5.2 Relative Longitudinal Displacement at Expansion Joints

Relative longitudinal displacements (RLD) at structural expansion joints, δ_L^{EXP}, are limited in order to prevent excessive rail axial stress. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

RLD at structural expansion joints, δ_L^{EXP}, has components due to both structural translation and structural rotation. For structural rotation, RLD is a function of distance from center of rotation to rail centroid. Therefore, δ_L^{EXP} shall be monitored relative to the original rail centroid location, and consist of structural movement alone. For δ_L^{EXP}, strains in the rail and rail slippage are not considered.

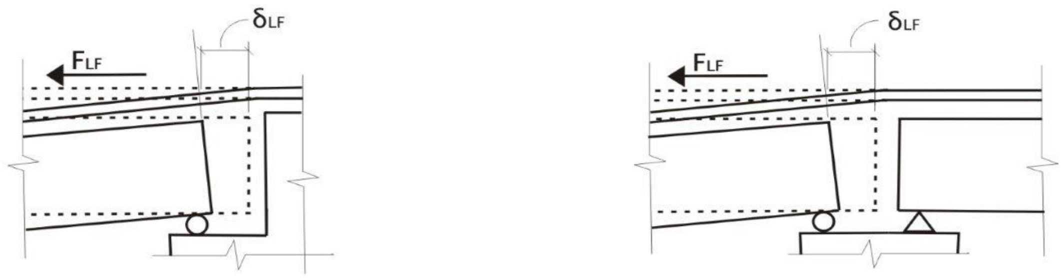
Limits to δ_L^{EXP} under Group 4 are the basis for initial proportioning of structures and to limit rail stress.

Limits to δ_L^{EXP} under Group 5 are to limit rail stress.

δ_L^{EXP}, consists of separate components:

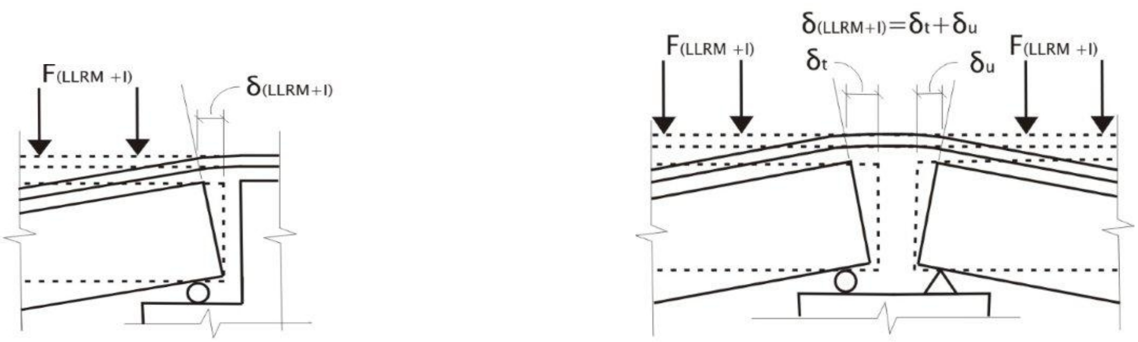
- δ_{LF} = component due to acceleration and braking only, see Figure 12-19.
- δ_{LLRM+I} = component due to vertical train plus impact loads only, see Figure 12-20.
- δ_{OBE} = component due to OBE only (see Figure 12-21), comprised of:
  - $\delta_{OBE(L)}$ = longitudinal displacement subcomponent due to OBE.
  - $\delta_{OBE(V)}$ = rotation about vertical axis subcomponent due to OBE.
  - $\delta_{OBE(T)}$ = rotation about transverse axis subcomponent due to OBE.
  - $\delta_{OBE} = \delta_{OBE(L)} + \delta_{OBE(V)} + \delta_{OBE(T)}$
- δ_{TD} = component due to temperature differential (T_D) between superstructure and rail.

Figure 12-19: δ_{LF} Definition



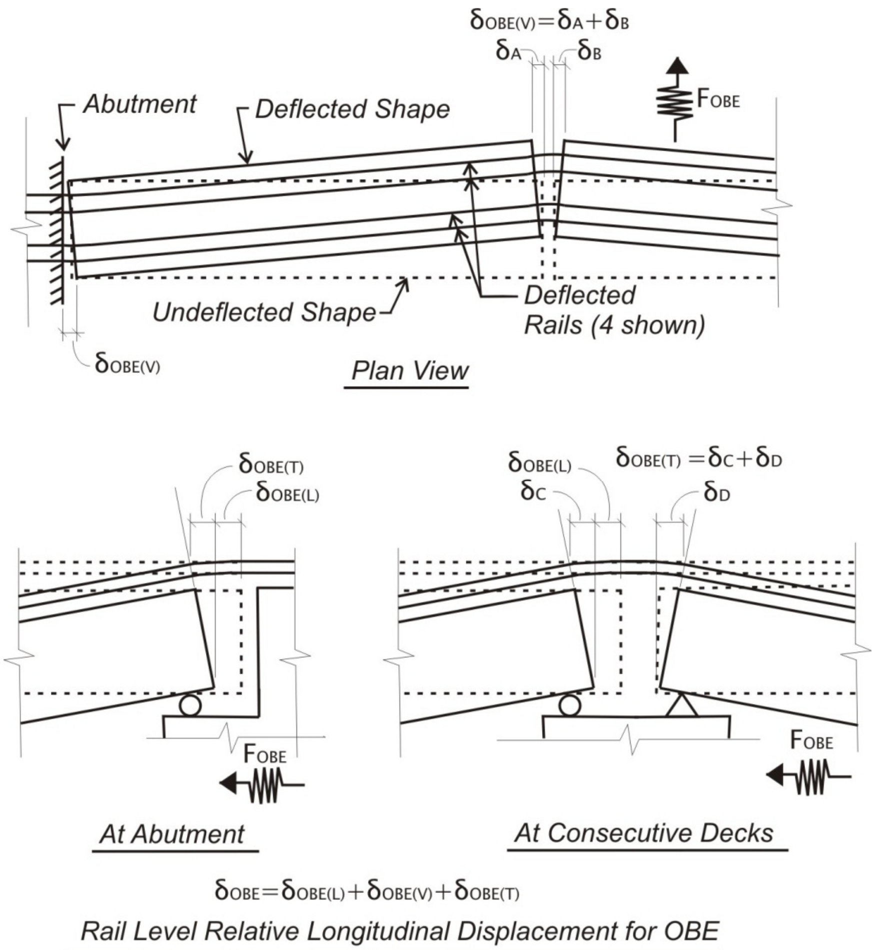
At Abutment *At Consecutive Decks*

Figure 12-20: δ_{LLRM+I} Definition



At Abutment *At Consecutive Decks*

Figure 12-21: δ_{OBE} Definition



The RLD at expansion joints measured relative to the original rail centroid location (δ_L^{EXP}) shall not exceed the limits given in Table 12-17.

Note that in order to prevent having separate load cases for relative displacement and rail stress design, the expected temperature differential demands are added to the displacement limits. The temperature differential demands are dependent on the structural thermal unit (L_{TU}), which is defined as the point from fixed point of thermal expansion to the next adjacent fixed point of thermal expansion. The maximum L_{TU} shall not exceed 330 feet.

Table 12-17: Relative Longitudinal Displacement at Expansion Joints Limits

Group	δ_L^{EXP} (inch)
4	$1.0 + \delta_{TD,Expected}$
5	$2.33 + 0.5\delta_{TD,Expected}$

Where:

$\delta_{TD,Expected}$ = expected RLD measured relative to the original rail centroid location due to T_D loading per Section 12.6.5.1. For most structures, $\delta_{TD,Expected}$ can be approximated:

$$\delta_{TD,Expected} = \alpha(\Delta T)L_{TU}$$

Where:

α = coefficient of thermal expansion for the superstructure

ΔT = 40°F temperature differential per Section 12.6.5.1. (ΔT always positive for calculation of $\delta_{TD,Expected}$)

L_{TU} = length of structural thermal unit at a given expansion joint. If both sides of expansion joint are longitudinally free to displace, then both adjacent spans must be considered in L_{TU}. The maximum L_{TU} shall not exceed 330 feet.

For any structure which $\delta_{TD,Expected}$ cannot be approximated with the above equation, including Complex Structures per *Seismic* chapter, $\delta_{TD,Expected}$ shall be verified by monitoring rail-structure interaction models subject to T_D loading per Section 12.6.5.1.

12.6.5.3 Relative Vertical Displacement at Expansion Joints

Relative vertical displacement (RVD) at structural expansion joints, δ_V^{EXP}, are limited in order to prevent excessive rail bending stress.

Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

The flexibility of the superstructure and substructure (i.e.: bearings, shear keys, columns, and foundations) shall be considered when calculating RVD.

The relative vertical displacement at expansion joints (δ_V^{EXP}), shown in Figure 12-22 shall not exceed the limits given in Table 12-18.

See Section 12.6.4.8 for RVD limits used to promote favorable track safety, passenger comfort, and track maintenance.

Figure 12-22: Relative Vertical Displacement at Expansion Joints

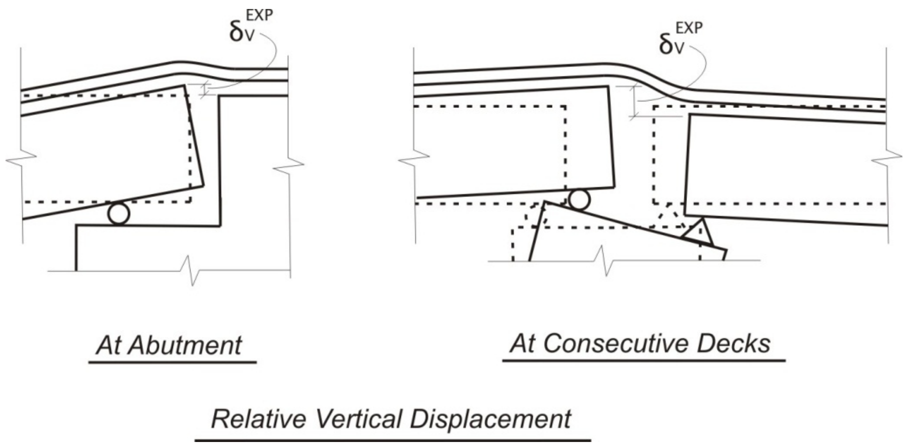


Table 12-18: Relative Vertical Displacement at Expansion Joints Limits

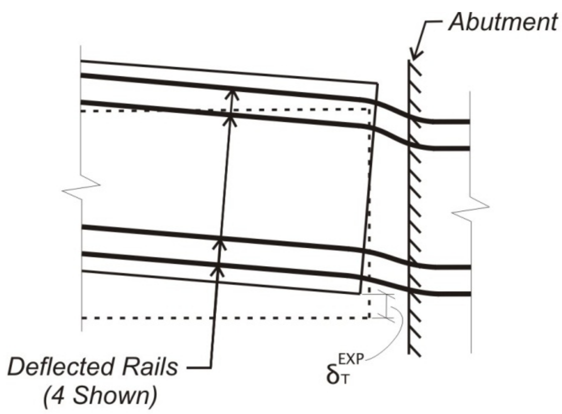
Group	δ_V^{EXP} (inch)
4	0.25
5	0.50

12.6.5.4 Relative Transverse Displacement at Expansion Joints

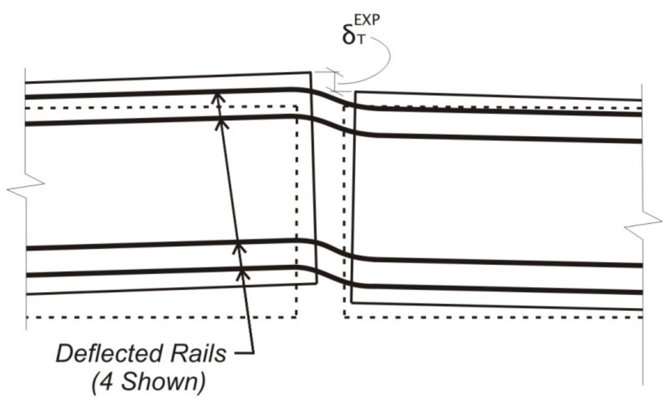
Relative transverse displacement (RTD) at structural expansion joints, δ_T^{EXP}, are limited in order to prevent excessive rail bending stress. Structural expansion joints between adjacent deck ends, and between deck ends and abutments shall be considered.

The relative transverse displacement at expansion joints (δ_T^{EXP}), shown in Figure 12-23, shall not exceed the limits given in Table 12-19.

Figure 12-23: Relative Transverse Displacement at Expansion Joints



At Abutment - Plan View



At Consecutive Decks - Plan View

See Section 12.6.4.9 for RTD limits used to promote favorable track safey, passenger comfort, and track maintenance.

Table 12-19: Relative Transverse Displacement at Expansion Joints Limits

Group	δ_T^{EXP} (inch)
4	0.08
5	0.16

12.6.5.5 Uplift at Direct Fixation Fasteners

For Groups 4 and 5, the direct fixation fastening system capacity, including the anchorage to supporting non-ballasted track, shall be designed to withstand calculated uplift force (F_{uplift}) by the factors of safety given in Table 12-20.

Table 12-20: Minimum Factor of Safety for Uplift on Direct Fixation Fasteners

Group	F_{uplift}
4	2.0
5	1.33

Specially designed fasteners with reduced vertical stiffness and/or increased uplift capacity may be required adjacent to structural expansion joints.

12.6.5.6 Permissible Additional Axial Rail Stress Limits

Permissible additional axial rail stress limits were developed considering total allowable rail stresses minus rail bending stresses due to vertical wheel loads and relative displacements at structural expansion joints. Additionally, the initial axial stress of the rail due to rail temperature and preheat during rail installation (per *Trackwork* chapter) was considered.

The permissible additional axial rail stress limits pertain to axial only rail stresses generated by track-structure interaction.

For rails on the bridge or aerial structure and adjacent abutment or at-grade regions, the permissible additional axial rail stresses (σ_{rail}) shall be per Table 12-21.

Table 12-21: Permissible Additional Axial Rail Stress Limits

Group	Range of σ_{rail}
4	-14 ksi ≤ σ_{rail} ≤ +14 ksi
5	-23 ksi ≤ σ_{rail} ≤ +23 ksi

Where negative is compression and positive is tension.

12.6.6 Dynamic Structural Analysis

Dynamic structural analysis of high-speed train passage (LLV) is required in order to determine resonancy induced dynamic impact (I_{LLV}) effects, and limit vertical deck accelerations. Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loading.

All dynamic structural analysis of high-speed train passage using actual high-speed trains shall consider the flexibility of superstructure and substructure (i.e: bearings, columns, and foundations).

To avoid over or underestimating the resonant speeds, two conditions must be investigated:

- Condition #1 – lower bound estimate of stiffness and upper bound estimate of mass.

- Condition #2 – upper bound estimate of stiffness and lower bound estimate of mass.

When evaluating vertical deck accelerations, Condition #2 – upper bound estimate of stiffness and lower bound estimate of mass, shall be investigated.

Modeling requirements for lower and upper bound estimates of stiffness and mass are given in Section 12.6.8.

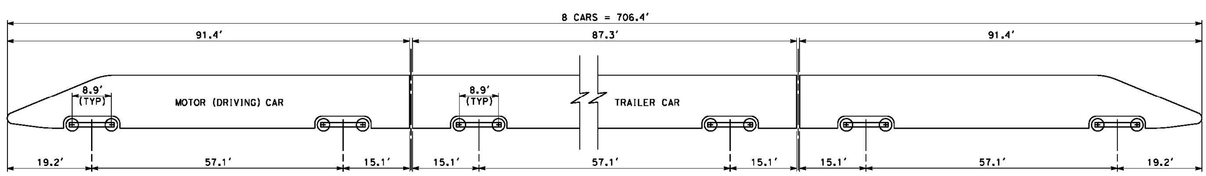
12.6.6.1 High Speed Train Loading (LLV)

Dynamic structural analysis of high-speed train passage shall consider representative trainsets (LLV), idealized as a series of moving vertical loads at specified axle and bogie spacings. Modeling of the train suspension system is not needed for dynamic structural analysis.

Five trainsets, shown in Figure 12-24 to Figure 12-28 collectively form LLV.

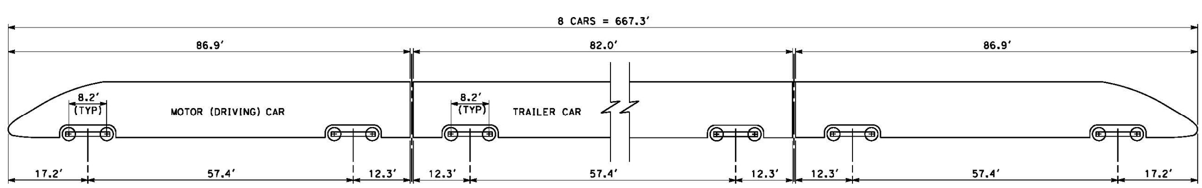
A full dynamic structural analysis using all five trainsets applies, subject to the suite of speeds given in Section 12.6.6.2.

Figure 12-24: Type 1



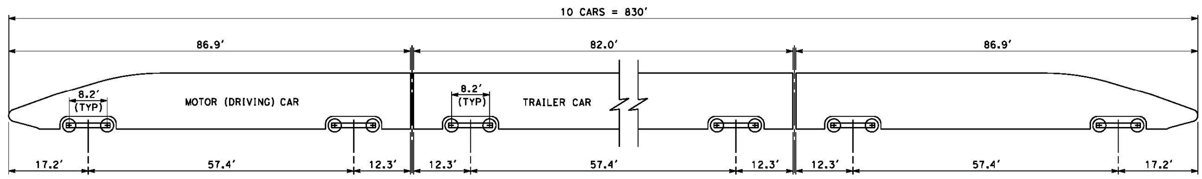
Maximum Axle Load = 18.7 tons Train Weight (Empty) = 510 tons

Figure 12-25: Type 2



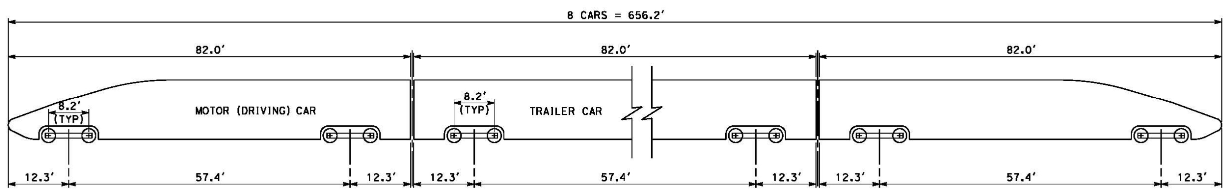
Maximum Axle Load = 16.5 tons Train Weight (Empty) = 529 tons

Figure 12-26: Type 3



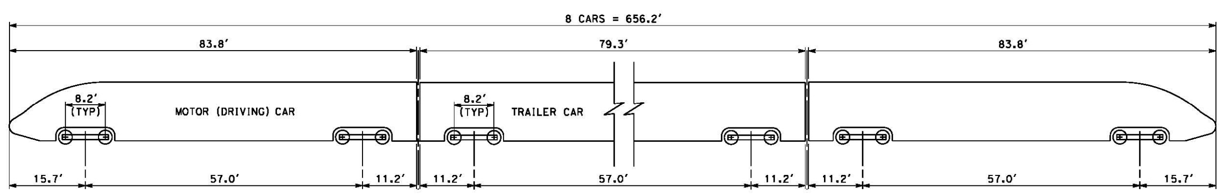
Maximum Axle Load = 12.95 tons Train Weight (Empty) = 500 tons

Figure 12-27: Type 4



Maximum Axle Load = 15.4 tons Train Weight (Empty) = 496 tons

Figure 12-28: Type 5



Maximum Axle Load = 18.7 tons Train Weight (Empty) = 493 tons

12.6.6.2 Train Speeds

A full dynamic structural analysis using all five trainsets applies, subject to the following suite of speeds:

- Speeds from 90 mph up to maximum speed of 1.2 times the line design speed (or 250 mph, whichever is less), by increment of 10 mph.
- Smaller increments of 5 mph for ±20 mph on each side of the first two resonant speeds.

A. <u>Resonant Speeds</u>

For simple spans, resonant speeds may be estimated by:

$V_i = n_o d/i$,

Where:

V_i = resonant speeds,

n_o = first natural frequency of vertical deflection

d = characteristic wheel spacing, see Figure 12-29

i = resonant mode numbers (e.g.: 1, 2, 3, 4, …)

Figure 12-29: Characteristic Wheel Spacing, d

For structures not consisting of simple spans, resonant speeds shall be determined by the dynamic structural analysis model.

B. <u>Cancellation Speeds</u>

In addition to resonance, cancellation effects also contribute to the overall dynamic response of elevated structures. For simple spans, cancellation speeds may be estimated by:

$$V_i = \frac{2\eta_o L}{2i - 1},$$

Where:

V_i = cancellation speeds,

n_o = first natural frequency of vertical deflection

L = simple span length

i = cancellation mode numbers (e.g.: 1, 2, 3, 4, …)

When L/d = 1.5, an optimal design condition exists for which the first mode of resonance aligns with the second mode of cancellation. In this condition, the primary dynamic residual response generated by repeated axle loads can be suppressed. Due to uncertainities associated with the service life of the structure, it may be unrealistic to design a given structure solely for a single characteristic wheel spacing. Nevertheless, optimal span lengths for potential trainsets shall be considered for design.

For more complex structures, the interaction between resonant and cancellation speeds may not be readily apparent and shall be investigated by a more detailed dynamic structural analysis.

12.6.6.3 Dynamic Impact Factors

For the high-speed trainsets (LLV), the dynamic model shall be used to determine the dynamic impact factors (I_{LLV}).

To determine (I_{LLV}), the maximum dynamic response value, ξ_{dyn} shall be found for each structural response for single track loading (LLV) over the range of speeds given in Section 12.6.6.2.

Compared against the corresponding static response value, ξ_{stat}, the dynamic impact factor is:

$$I_{LLV} = \max\left[\frac{\xi_{dyn}}{\xi_{stat}}\right]$$

12.6.6.4 Vertical Deck Acceleration

Vertical acceleration of bridge and aerial structure decks are limited to avoid reduction in wheel contact, and passenger discomfort.

When evaluating vertical deck accelerations, an upper bound estimate of stiffness and lower bound estimate of mass, shall be considered.

Vertical acceleration of bridge and aerial structure decks shall be found for single track loading (LLV) over the range of train speeds given in Section 12.6.6.2.

The maximum vertical deck acceleration shall be limited to 16.1 ft/s^2 (0.50g).

Note that this pertains to accelerations at the top of structural deck. For acceleration limits to be experienced within the train car body, see Section 12.6.7.

12.6.7 Dynamic Vehicle-Track-Structure Interaction Analysis

Should a structure fall within the recommended vertical frequency range (Section 12.6.3.1), then dynamic vehicle-track-structure interaction (VTSI) analysis shall not be required.

Should a structure fall outside of the recommended vertical frequency range (Section 12.6.3.1), then dynamic VTSI shall be required.

For Complex Structures per *Seismic* chapter, dynamic VTSI analysis may be required, as determined by the approved Seismic Design and Analysis Plan.

For typical structures, limiting the vertical, transverse, and torsional frequencies of the span, span deflections, relative displacements between spans, expansion joint widths, and deck acceleration provides sufficient guidance for track safety and passenger comfort. However, an advanced VTSI analysis is required for structures operating outside the known limits of acceptable performance, or structures with untested design concepts.

The purpose of VTSI analysis is to verify track safety and passenger comfort by considering the interaction between the vehicle, track, and structure.

Track safety depends primarily upon the contact forces between the rail and the wheel. The ratio of lateral to vertical forces (L/V ratio) is typically used as the primary indicator of derailment. In addition, the magnitudes of lateral and vertical forces imparted by the wheel to the rail must be controlled.

Passenger comfort depends primarily upon the the accelerations experienced within the train car body during travel on and off bridge or aerial structures.

12.6.7.1 Dynamic Train-Structure Interaction Analysis Requirements

For dynamic VTSI, both a dynamic structural model and dynamic trainset models shall be used. The interaction of the structure and trainset models shall be considered in either a coupled or iterative method.

Details of structural modeling requirements are given in Section 12.6.8.

Due to uncertainty of trainset selection, multiple trainset models shall be proposed for dynamic VTSI. Each of the dynamic trainset models shall be consistent with characteristic loading of LLV trainsets as defined in Section 12.6.6.1, and consider the mass, stiffness, and damping characteristics of the wheels, bogies, suspension, and body.

It has been shown that vehicle response is highly sensitive to track irregularities. For dynamic VTSI analysis, random track irregularities shall be considered directly in the VTSI model. Random theoretical irregularities shall be developed for FRA Track Classes using a power spectral density function which may be distributed into the time domain by applying the spectral representation method.

Dynamic VTSI analysis shall consider a series of speeds ranging from a minimum of 90 mph up to maximum speed of 1.2 times the line design speed (or 250 mph, whichever is less).

Dynamic VTSI analysis shall consider single track (i.e., one trainset) loading only.

For the dynamic VTSI analysis, a sufficient number of cars shall be used to produce maximum load effects in the longest span of the structure. In addition, a sufficient number of spans within a long viaduct structure shall be considered to initiate any resonance effects in the train suspension.

12.6.7.2 Dynamic Track Safety Criteria

Dynamic track safety criteria shall not exceed the limits given in Table 12-22 for any trainset across the required speed range.

Table 12-22: Dynamic Track Safety Limits

Parameter	Dynamic Track Safety Criteria
Maximum Single Wheel L/V Ratio	$L/V_{wheel} \leq 0.80$
Maximum Truck Side L/V Ratio	$L/V_{truck\ side} \leq 0.6$
Minimum Single Wheel Dynamic Vertical Load	$V_{wheel,dynamic} \geq 0.15*V_{wheel,static}$
Maximum Net Axle Dynamic Lateral Force	$L_{axle,dynamic} \leq 0.40*V_{axle,static} + 5$ kips

Where:

L/V_{wheel} = Ratio of lateral forces to vertical forces exerted by a single wheel on the rail

$L/V_{truck\ side}$ = Ratio of lateral forces to vertical forces exerted by any one side of a truck (bogie) on the rail

$V_{wheel,dynamic}$ = Dynamic vertical wheel reaction

$V_{wheel,static}$ = Static vertical wheel load

$L_{axle,dynamic}$ = Dynamic lateral axle reaction

$V_{axle,static}$ = Static vertical axle load

12.6.7.3 Dynamic Passenger Comfort Criteria

The maximum lateral acceleration within the car body is limited to 1.6 ft/s^2 (0.05 g) for any trainset across the required speed range.

The maximum vertical acceleration within the car body is limited to 1.45 ft/s^2 (0.045 g) for any trainset across the required speed range.

12.6.8 Modeling Requirements

The following modeling requirements for static and dynamic analysis of high-speed train bridge and aerial structures are required for project-wide consistency.

12.6.8.1 Model Geometry and Boundary Conditions

The model shall represent the bridge or aerial structure's span lengths, vertical and horizontal geometries, column heights, mass and stiffness distribution, bearings, shear keys, column or abutment supports, and foundation conditions.

For isolated bridges, with no adjacent structures, the model shall represent the entire bridge including abutment support conditions.

For repetitive aerial structure viaducts with simply supported spans the model shall have a minimum of 20 spans. Boundary conditions at the ends of the model shall represent the stiffness of any adjacent spans or frames.

For repetitive aerial structure viaducts with continuous span frames (i.e., each frame consists of multiple spans with moment transfer between the deck and columns), the model shall have a minimum of 5 frames. Boundary conditions at the ends of the model shall represent the stiffness of adjacent spans or frames.

Soil springs at the foundations shall be developed based on reports defined in the *Geotechnical* chapter.

For modeling of earthern embankments or cuts at bridge approaches, see Section 12.6.8.7.

12.6.8.2 Model Stiffness

Structural elements shall be represented by the appropriate sectional properties and material properties.

For frequency analysis and dynamic structural analysis, and dynamic VTSI analysis, both upper and lower bound estimates of stiffness shall be considered.

For track serviceability and rail structure interaction analysis, a lower bound estimate of stiffness shall be considered.

For steel superstructure and column members:

- Upper bound stiffness: full steel cross sectional properties, and expected material properties (larger than nominal specified per CBDS) shall be used.
- Lower bound stiffness: reduced steel cross sectional properties considering shear lag effects if necessary, and nominal material properties shall be used.

For reinforced, pre-stressed, and post-tensioned concrete superstructure members:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used. Consideration shall be made for composite action of the superstructure with non-ballasted track, barriers or derailment walls when determining upper bound bending inertias.
- Lower bound stiffness: effective bending inertia, I_{eff}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

For concrete column members:

- Upper bound stiffness: full gross bending inertia, I_g, and modulus of elasticity corresponding to expected material properties (1.3x nominal) per CSDC shall be used.

- Lower bound stiffness: cracked bending inertia, I_{cr}, per CSDC, and modulus of elasticity corresponding to nominal material properties shall be used.

As an alternative to using I_{cr} per CSDC, an effective bending inertia, I_{eff}, which considers the maximum moment demand, M_a, and the cracking moment, M_{cr}, may be used in accordance with AASHTO. Also, a moment-curvature representation of the column stiffness may be used.

12.6.8.3 Model Mass

For frequency analysis, dynamic structural analysis, and dynamic VTSI analysis, both upper and lower bound estimates of bridge mass shall be considered.

For track serviceability and rail structure interaction analysis, an upper bound estimate of bridge mass shall be considered.

For structural dead load (DC) mass, the material unit weights per Section 12.5 shall be used as the basis for design. For the upper bound mass estimate, unit weights shall be increased by a minimum of 5 percent. For the lower bound mass estimate, unit weights shall be reduced by a minimum of 5 percent.

For superimposed dead load (DW), upper and lower bound mass estimates shall be considered.

12.6.8.4 Model Damping

When performing OBE time history analyses for track serviceability and rail-structure interaction analysis, damping per *Seismic* chapter shall be used.

When performing dynamic structural analysis, the peak structural response at resonant speed is highly dependent upon damping. The damping values in Table 12-23 shall be used.

Table 12-23: Damping Values for Dynamic Model

Bridge Type	Percent of Critical Damping
Steel and composite	0.5%
Pre-stressed, post-tensioned concrete	1.0%
Reinforced concrete	1.5%

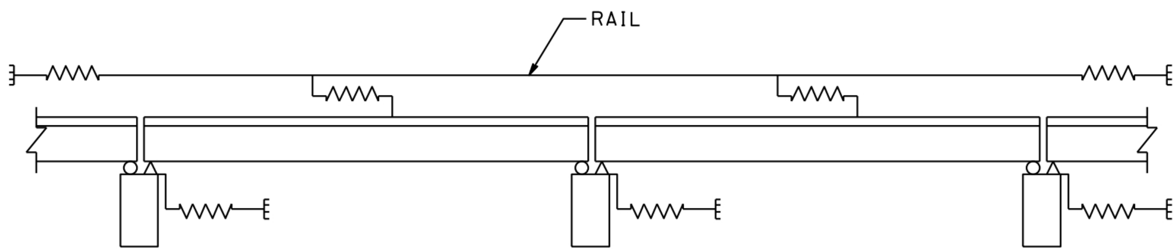
The damping may be increased for shorter spans (< 65 feet), with supporting evidence by Designer.

When performing dynamic structural analysis using actual high-speed trains, soil damping shall be considered in accordance with the Geotechnical reports described in the *Geotechnical* chapter.

12.6.8.5 Modeling of Rail-Structure Interaction

Longitudinal actions produce longitudinal forces in the continuous rails. These forces are distributed to the bridge and aerial structures in accordance with the relative stiffness of the non-ballasted track and fasteners, articulation of the structural system, and stiffness of the substructure, see Figure 12-30 for a schematic rail-structure interaction model.

Figure 12-30: Rail-Structure Interaction Model



Rail-structure interaction may govern the:

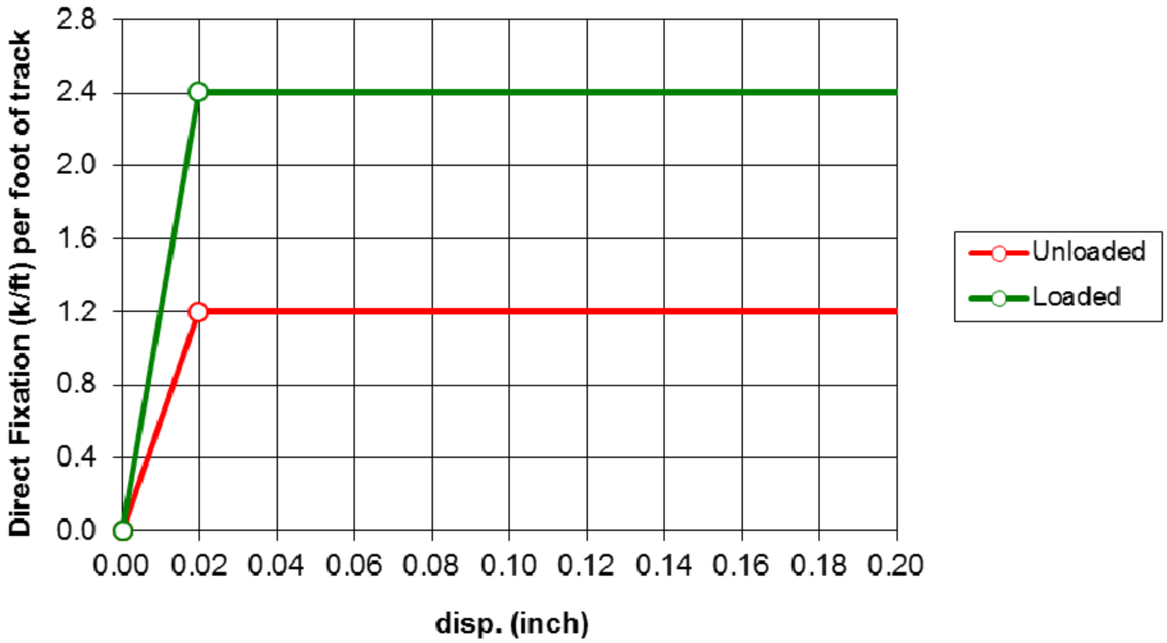
- Location and distance between bridge expansion joints.
- Stiffness of the bridge superstructure.
- Stiffness of the supporting columns and foundations.

Rail-structure interaction shall be performed for all structures using either static or dynamic models. In addition, the model shall include the stiffness of the rails appropriately located upon the superstructure, and longitudinal bi-linear coupling springs between the track and superstructure over the length of the model.

For purposes of this analysis, the rail section shall be 141 RE per AREMA. In the event that another rail section is considered, a special rail-structure interaction analysis is required per Section 12.6.8.6.

Fastener restraint is nonlinear to allow slippage of the rail relative to the track support structure. The bi-linear coupling springs shall represent non-ballasted track with direct fixation fasteners (see Figure 12-31) between the rails and superstructure on a per track basis. This relationship represents a pair of fasteners with 1.35 kip (6 kN) unloaded longitudinal restraint at 27-inch spacing.

Figure 12-31: Non-Ballasted Track with Direct Fixation Fasteners: Bi-linear Coupling Springs



Assumed fastener properties represent a uniform distributed longitudinal restraint of 1.2 k/ft (unloaded) per foot of track. In practice, variations in fastener spacing may be required to accommodate structural expansion joints, deck skew, or other geometric constraints.

Uniform longitudinal restraint shall be verified using the following uniformity criteria:

Distributed longitudinal restraint calculated for fastener locations over any 10 ft. length of track along the structure shall be within +/-20% of the assumed uniform bi-linear coupling relation.

For aerial structures that meet the uniformity criteria, but not consistent with Figure 12-31, the structure shall be considered to have a nonstandard fastener configuration (NSFC). These structures require an approved design variance and special rail-structure interaction analysis per Section 12.6.8.6.

For aerial structures that do not meet the uniformity criteria, the structure shall be considered to have a non-uniform fastener configuration (NUFC). These structures require an approved design variance and a special rail-structure interaction analysis per Section 12.6.8.6.

The total number of longitudinal bi-linear coupling springs per each span shall not be less than 10 and the spacing between the springs shall not be more than 10 feet.

For vertical stiffness of fasteners, 4000 k/ft per foot of track (pair of rails) shall be used. For purposes of evaluating this design criteria, constant vertical stiffness shall be used to model fastener compression and tension (uplift).

For lateral (i.e., transverse) stiffness of fasteners, 450 k/ft per foot of track (pair of rails) shall be used.

12.6.8.6 Special Rail-Structure Interaction Analysis

Rail-structure interaction limits are developed considering typical fastener configurations on typical structures. For those systems that do not meet these assumptions, new limits must be developed using a refined analysis.

A special rail-structure interaction analysis shall be required for those structure and track designs requiring a design variance. Specific design variances requiring special rail-structure interaction analysis include, but are not necessarily limited to: designs requiring nonstandard fastener configurations (NSFC), non-uniform fastener configurations (NUFC), rail section other than RE 141, structures with thermal units (L_{TU}) greater than 330 feet, rail expansion joints (REJs), and ballasted track on aerial structures.

The Contractor shall identify and document structure types requiring special rail-structure interaction analysis as part of the Type Selection process described in Section 12.8.1.1. After completion of Type Selection and upon determination that the selected structure type requires a special rail-structure interaction analysis, the Contractor shall develop a Rail Stress and Fasteners Design and Analysis Plan (RSFDAP) as part of the design variance submittal. The RSFDAP shall formally identify elements requiring special consideration, including but not limited to: refined fastener properties, detailed temperature analysis, refined ballast/nonballasted properties, and rail expansion joint locations. A detailed proposal of analysis procedures used to verify track performance (including track safety, passenger comfort, track maintenance, and rail stress) shall be submitted as part of the RSFDAP.

Examples of special analysis required may include, but is not limited to: development of new rail-structure interaction limits, development of new analytical model elements, local rail stress modeling, site-specific temperature analysis, analysis of impacts to track maintenance, etc.

12.6.8.7 Modeling of Rail-Structure Interaction at Model Boundaries

Where an abutment occurs at the ends of bridges and aerial structures, the rails and coupling spring fastener elements shall be extended a distance of L_{ext} from the face of the abutment. At the model boundary (i.e., at L_{ext} from abutment), a horizontal boundary spring representing the rail/fastener system behavior shall be used. The boundary spring, which represents unloaded track, shall be elastic-perfectly plastic, with a elastic spring constant of k (k/ft) yielding at P_b (kips), which represents the maximum capacity of an infinite number of elastic fasteners.

The yielding of the boundary spring at P_b is a threshold value that should be checked throughout the track-structure interaction analysis. If at any point during the analysis the boundary spring yields at force P_b, L_{ext} should be increased and the analysis should be repeated until elastic boundary spring behavior is verified.

The boundary spring behavior depends on the at-grade track type and fasteners. Values of k, P_b, and L_{ext} are given for a variety of direct fixation fasteners and track types in Table 12-24. Note that the minimum recommended values of L_{ext} are dependent on the average span length of the bridge or aerial structure (denoted L_{avg}):

$$L_{avg} = \frac{(L_1 + L_2 + ... + L_n)}{n} \quad \text{= the average span length}$$

Table 12-24: Minimum Recommended Track Extension and Boundary Spring Properties

Non-Ballasted Track (fasteners yield at 0.02 inches)			
Yield Load per foot of non-ballasted track	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.20 kips per foot of track [1.35 kips (6 kN) fasteners @ 27” o.c.]	24,200	40.3	$0.1L_{avg}$ + 350
1.80 kips per foot of track [2.02 kips (9 kN) fasteners @ 27” o.c.]	29,600	49.3	$0.1L_{avg}$ + 275
2.40 kips per foot of track [2.70 kips (12 kN) fasteners @ 27” o.c.]	34,200	57.0	$0.1L_{avg}$ + 240
Ballasted Track (fasteners yield at 0.08 inches)			
Yield Load per foot of ballast	**k (kips/ft)**	**P_b (kips)**	**Min. Recommended L_{ext} (feet)**
1.37 kips/ft of track	12,900	86.0	$0.1L_{avg}$ + 310

In the event that an additional bridge or other elevated structure is located within the L_{ext} model boundary distance from the face of an earthen abutment, the additional structure (including the loads and modeling requirements presented in this section) shall also be included in the track-structure analysis model.

Assumptions used to develop Table 12-24 are expected to apply for the majority of elevated structures and viaducts, which are assumed to be in simply-supported configuration with uniform distribution of fasteners. For alternative structure and fastener configurations, additional investigation shall be required to appropriately define the model boundary.

12.6.8.8 Modeling of Earthen Embankments or Cuts at Bridge Approaches

Where applicable under rail-structure interaction Group 5 load cases, the vertical and lateral stiffness of slab or ballasted track upon earthen embankments or cuts shall be determined to accurately predict relative displacements at abutment expansion joints, and rail stress at the abutment and at-grade regions.

For vertical stiffness of track upon earthen embankment or cuts, a minimum value of 350 pci shall be used in accordance with AREMA subgrade requirements.

For lateral (i.e., longitudinal and transverse) stiffness of track upon earthen embankments or cuts, consideration of embankment flexibility, non-ballasted track or ballast tie embedment, passive pressure, and friction shall be made in accordance with the Geotechnical reports described in the *Geotechnical* chapter.

OBE ground motions shall be applied concurrently at structural foundations and earthen embankments or cuts to prevent incompatibility between the vibrating structure and the relatively stationary track upon earthen embankment or cut. For tall embankments or specific soil types, lag times and/or amplification effects shall be considered for OBE ground motions in accordance with the Geotechnical reports described in the *Geotechnical* chapter.

12.7 Structural Design of Surface Facilities and Buildings

The static design of stations, surface facilities, buildings and ancillary structures shall conform to the requirements of CBC, AISC, and ASCE 7 whichever is applicable except as specified otherwise. The seismic design of stations, surface facilities, buildings, and ancillary structures that are primary structures shall conform to the requirements of CBC, AISC, and ASCE 7, whichever is applicable. In addition, the seismic requirements of the *Seismic* chapter shall apply.

For description of Primary and Secondary structures as well as seismic requirements, refer to the *Seismic* chapter.

Primary structures supporting HSTs shall meet the requirements of Section 12.5 for Permanent and Transient Loads for Structures Supporting HST. If such a facility has uses other than supporting or in addition to HSTs or other types of bridges it shall also meet the requirements of this Section 12.7. For foundation design, see the requirements in the *Geotechnical* chapter.

12.7.1 Load Requirements for Stations, Surface Facilities and Buildings

Elevated and at-grade station structures not supporting HSTs, as well as surface facilities and buildings, shall be subject to CBC requirements, with additional criteria specific to the HST.

Station platforms, mezzanines, and aerial pedestrian access ramps shall be subject to additional criteria specific to the CHSTP herein.

12.7.1.1 Dead Load and Superimposed Dead Load

Dead load and superimposed dead load shall include but not be limited to the following:

- Dead weight of structural members and architectural finishes,
- Dead weight of road surface and of backfill above the structures,
- Dead weight of surcharge loads,
- Dead weight of equipment and appurtenances.

Refer to Section 12.5 on Permanent and Transient Loads for Structures Supporting HST for the unit weights of materials.

12.7.1.2 Train Load

Refer to Section 12.5 on Permanent and Transient Loads for Structures Supporting HST for the train loading.

12.7.1.3 Roof Load

Roof live load and reduction factors shall be in accordance with the CBC.

12.7.1.4 Floor Load

Floor live load shall be in accordance with the CBC with no reduction in floor live load, except for parking structures.

Station platforms and concourse areas shall be designed for a floor live load of 100 psf.

Emergency and maintenance walkways shall be designed for a floor live load of 100 psf.

Floor live loads on service walkways and sidewalks shall be designed for a live load of 100 psf, or a concentrated load of 2,000 pounds applied anywhere on the walkway and distributed over a 4 feet by 2 feet area.

The structural system supporting the access doors at street level shall be designed for a floor live load of 350 psf.

Storage area floor live loads shall be 100 psf.

Areas where cash carts are used shall be designed to accommodate a point live load of 350 pounds per wheel. Wherever station configuration requires that cash carts cross pedestrian bridges, bridges shall be designed to accommodate this live load.

Operations Control Centers shall be designed for a floor live load of 100 psf.

Equipment room floors for such uses as signals, communications, power, transformers, battery storage and fanrooms shall be designed for a minimum floor loading sufficient to support both 350 psf distributed load and a 2000 pounds concentrated load or the actual equipment weight located so as to produce the maximum load effects in the structural members.

Pump rooms, service rooms, storage space, and machinery rooms shall be designed for floor live load of 250 psf, to be increased if storage or machinery loads so dictate.

Stairways shall be designed for a floor live load of 100 psf or a concentrated load of 300 pounds on the center of stair treads, whichever is critical. Impact shall not be considered for stairways.

Maintenance buildings will require overhead cranes and crane rails or floor mounted hydraulic jacks to lift individual cars from trains. The car loads are unknown until a vehicle is selected. The Designer shall coordinate with the Authority to obtain design requirements for crane design.

12.7.1.5 Vehicular Load

Parking areas for automobiles shall be designed to a minimum load as specified in the CBC. Bus load shall be designed to carry HL-93 loading in accordance with AASHTO LRFD with Caltrans Amendments.

Gratings in areas that are subject to vehicular loading shall be designed to carry HL-93 loading.

12.7.1.6 Miscellaneous Loads

Pedestrian safety railings shall be designed to withstand a horizontal force of 50 pounds per linear foot applied at any angles to the top of the railing. The mounting of handrails and framing of members for railings shall be such that the completed handrail and supporting structure shall be capable of withstanding a load of at least 200 pounds applied in any direction at any point on the top rail. These loads shall not be combined with the 50 pounds per linear foot. For the design of structure components that support train loads and a walkway, the walkway live loads shall not be applied simultaneously with the train loads.

Stationary and hinged cover assemblies internal to HST facilities shall be designed for a minimum uniform live load of 100 psf or a concentrated live load of 1,000 pounds over a 2 feet by 2 feet area. Deflection at center of span under 100 psf load shall not be more than 1/8 inch.

Gratings in sidewalks and in areas protected from vehicular traffic shall be designed for a uniform live load (LL) of 300 psf.

12.7.1.7 Slipstream Effects from Passing Trains

Refer to Section 12.5.2.7 for slipstream effects from passing high-speed trains.

Where structural elements can also be subjected to wind load, loading due to the slipstream effects from passing trains shall be considered to occur in combination with wind load.

Where trains are enclosed between walls and with a ceiling and deck, the design requirements for tunnels shall be met (see *Tunnels* chapter) including the following:

- Minimum cross section area of the through trackway
- Evacuation
- Fire/Life Safety
- Medical Health Criteria

In addition, transient air pressure analyses (as in a tunnel ventilation analysis) shall be used to determine the maximum transient air pressure acting on the walls and ceiling. These pressures shall be used for design of those elements such as uplift of ceilings or lateral pressure on walls and doors.

12.7.1.8 Seismic Design for Stations and Ancillary Facilities

Seismic design of primary and secondary structure facilities as defined in the *Seismic* chapter shall comply with the requirements of the *Seismic* chapter.

12.7.1.9 Collision Loads in Stations

Columns in stations shall be classified into three groups, according to the following criteria:

- GROUP A – This group consists of columns where the clearance measured from the TCL to face column and relevant conditions are as follows:
  - $\geq$ 16.5 feet
  - < 16.5 feet and within the station platform area provided that the platform is of massive construction and the platform edge is at least 1.25 feet above the level of the nearest rail.

  No collision impact forces shall be applied.
  - GROUP B – GROUP B columns are those located in a row of columns which run adjacent and parallel to the HST track and which do not meet the criteria of GROUP A. Columns in the row are classified as GROUP B, with the exception of the first and last ones. The column row shall include a column protection wall throughout its length. . The performance for this loading is a no collapse requirement.
  - The column protection wall shall comprise a lower guide wall together with an upper guide beam as shown in Figure 12-32. Due to the presence of the column protection wall, the GROUP B columns need not withstand full face collisions, but only grazing impacts by trains that have already been derailed. The lower guide wall and the upper guide beam shall be designed to withstand collision impact loads.
  - Columns and column protection walls shall be designed for one of the following horizontal collision impact loads, whichever produces the most adverse effect:
  - Columns shall be designed to resist a 900 kip force parallel with the TCL acting together with a 400 kip force at 90 degrees to the TCL, both 4 feet above low rail level and 225 kip force at 90 degrees to the TCL, 10 feet above TOR.
  - Lower guide wall shall be designed to resist a 900 kip force parallel with the TCL acting together with a 400 kip force at 90 degrees to the TCL, both 3.5 feet above top of low rail.
  - Upper guide beam shall be designed to resist a 350 kip force at 90 degrees to the TCL, acting 10 feet above top of low rail.
- GROUP C – Group C consists of the first and last columns in a row that do not belong to Group A or Group B.
  - The collision loads for each group of columns, as indicated above, are as follows:

- o Columns shall be designed for one of the following horizontal collision impact loads, whichever produces the most adverse effect. The performance for this loading is a no collapse requirement. A 2250 kip force parallel with the TCL acting together with an 800 kip force at 90 degrees to the TCL, both acting 4 feet above top of low rail
- o A 225 kip force at 90-degree to the TCL, acting 10 feet above low rail level

Alternatively, a protection device designed to resist the GROUP C impact loads shall be provided at the open face of the column as shown in Figure 12-33. The column in this figure shall be designed for the GROUP B column impact loads.

Figure 12-32: Collision Loads for Each Group of Columns

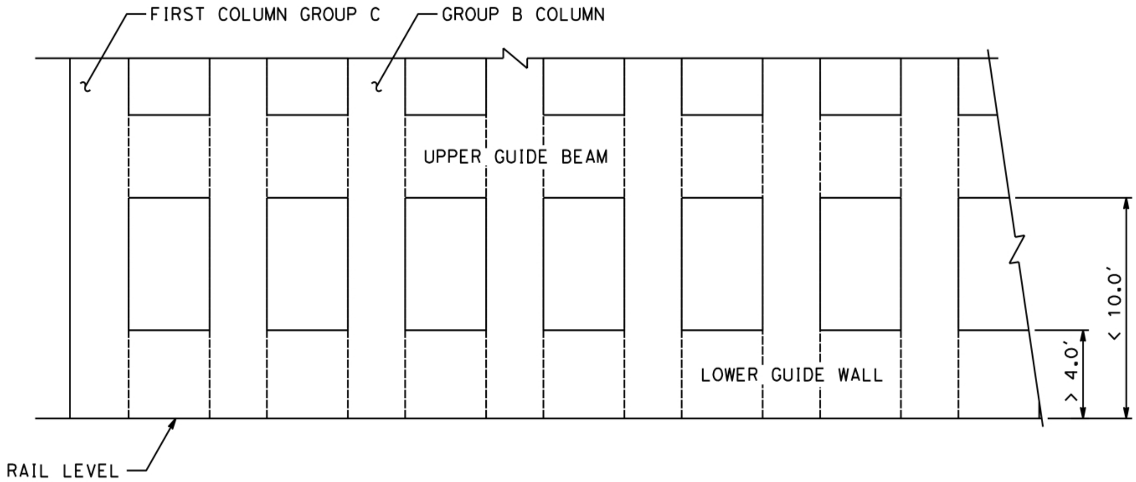
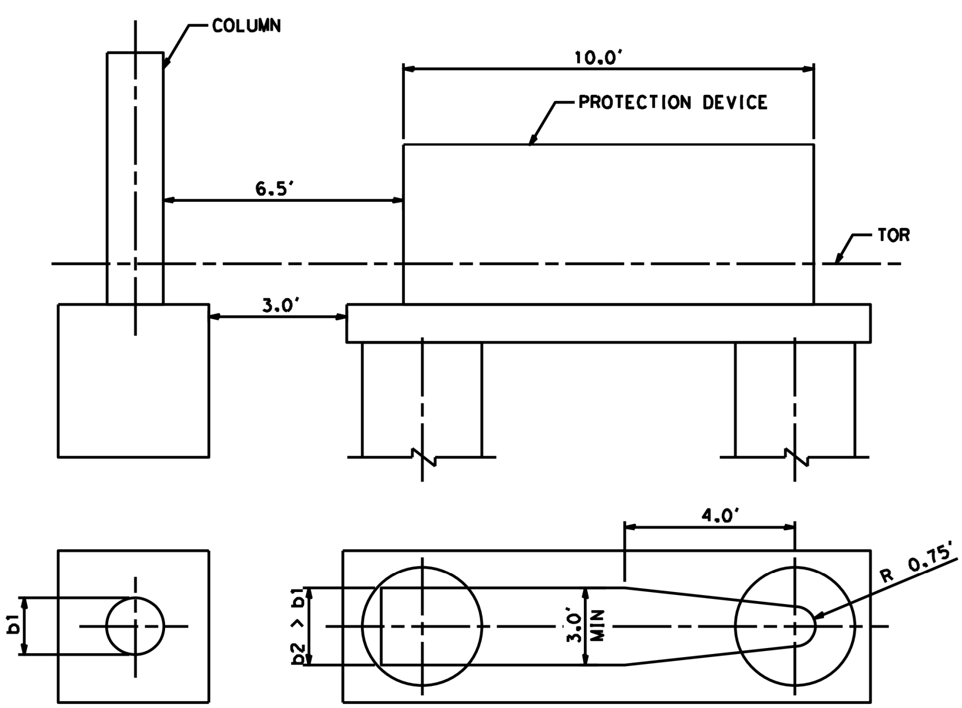


Figure 12-33: Protection Device



12.7.1.10 Collision Loads on Platforms

Platforms shall be designed to withstand a horizontal collision impact load of 225 kips applied at 90-degree to the TCL of the nearest track located anywhere along the platform.

A 1-foot-wide void shall be provided around columns that are within platform areas to prevent transfer of collision loads to the column.

12.7.1.11 Wind Loads

Wind loads including both windward and leeward sides of buildings and other structures shall be in accordance with the provisions of CBC, with $I_w = 1.15$.

12.7.1.12 Effects of Temperature, Shrinkage and Creep

Effects of temperature, shrinkage and creep shall be considered for structures above ground, as per requirements of the CBC.

12.7.1.13 Frequency and Vibration Limits

Station structures shall be designed to meet the following requirements for pedestrian comfort:

- The comfort criteria shall be defined in terms of maximum acceptable acceleration of any part of the station platform or deck occupied by the public. The following accelerations are the recommended maximum values for any part of the station platform or deck:

- 2.3 ft/s^2 for vertical vibrations
- 0.7 ft/s^2 for horizontal vibrations
- 1.3 ft/s^2 for exceptional crowd conditions vertical vibrations

- A verification of the comfort criteria shall be performed if the fundamental frequency of the deck is less than:
  - 5 Hz for vertical vibrations
  - 2.5 Hz for horizontal (lateral) and torsional vibrations. Transverse frequency analysis shall consider the flexibility of superstructure <u>only</u>, excluding the flexibility of bearings, columns, and foundations, assuming the supports at the ends of the span are rigid.

12.7.2 Foundations for Equipment Enclosures

Refer to the *Traction Power Supply System* chapter, *Automatic Train Control* chapter, and the *Communications* chapter.

For other equipment facilities, follow geotechnical recommendations and the provisions of CBC for design of foundations.

12.7.3 Foundations for Utility Equipment

Foundations for utility equipment shall comply with the requirement of CBC and in addition meet the requirements of the individual utility.

12.8 Design Considerations for Bridges and Aerial Structures

Unless otherwise specified, aerial structure design of HST structures as well as highway bridge structures shall be performed in accordance with AASHTO LRFD with Caltrans Amendments.

Design of HST aerial structures shall satisfy criteria that exceed those of highway and normal rail bridges because of the following:

- Particular effects which are critical in HST aerial structures:
  - Frequency of repetition (fatigue of materials)
  - Repetitive load applications (dynamic structural response)
  - Interaction of track and structure
- Riding comfort criteria
- High operating demands (life time of structure)

- Limited hours available for inspection, maintenance and repair.

To meet the above mentioned criteria, HST aerial structures shall be designed to conform with the following characteristics:

- Small deflections and good resilience to dynamic responses to ensure passenger safety and a very high level of comfort
- Low probability of resonance
- Conceptual simplicity and standardization for ease of construction, schematic quality control, fast track construction and higher maintenance reliability
- Reduction of environmental noise and vibration impact.

12.8.1 General Design Requirements

12.8.1.1 Type Selection

The type of bridges selected for design and construction shall be selected in a type selection process. This process is best described in the requirements of the Caltrans OSFP Information and Procedures Guide. The Authority's preference for type is prestressed concrete single box girders carrying two tracks for main line aerial trackways and structures. Box girders can be precast, precast segmental, or cast in-place, cast-in-place span by span or other similar types of construction. For specific locations, thru girders or thru trusses constructed by piece or incrementally launched may be more appropriate. The Type Selection shall include seismic considerations, foundation recommendations, aesthetics review, traffic maintenance (both highway and rail), drainage considerations and intrusion protection. Included with or in the Type Selection Report shall be all of the following that apply to the specific bridge or aerial structure:

- Type Selection Memo (see Caltrans OSFP Information and Procedures Guide)
- Hydrology and Hydraulics reports
- Aesthetics Design and Review Report
- Geotechnical Engineering Design Report
- Seismic Design and Analysis Plan (*Seismic* chapter)
- Rail Stress and Fastener Design and Analysis Plan (Section 12.6.8.6)
- Complex and Non-Standard Aerial Structures Load Path Report (Section 12.8.7)

If rail expansion joints are considered the variance process should start at Type Selection.

For isolated structures and the structures not in the main line, and highway bridges, structure type selection shall follow the requirements of the Caltrans OSFP Information and Procedures Guide. The Type Selection shall be coordinated with the Authority to determine and identify any constraints that may control the design that are not listed in these Design Criteria.

12.8.1.2 Clearances

Clearances requirements are specified in *Trackway Clearances* chapter.

12.8.1.3 Water Crossings

Hydraulic requirements for bridge drainage and requirements for water crossings are specified in the *Drainage* chapter.

12.8.1.4 Deck Arrangement

The arrangement of deck features shall conform to the requirements presented in the Standard and Directive Drawings.

12.8.1.5 Material Requirements

A. Concrete Requirements

The minimum 28-days concrete compressive strength in aerial structures shall be as follows:

- For piles, shafts, and footing reinforced concrete cast-in-place structures, f'c = 4,000 psi.
- For above ground reinforced concrete cast-in-place structures, f'c = 5,000 psi.
- For cast-in-place prestressed concrete, f'c = 6,000 psi.
- For precast prestressed members, f'c = 6,000 psi.
- Lightweight concrete is not allowed in Primary Type 1 structures. It may be used in secondary concrete such as leveling or non-ballasted track concrete.

For design of cast-in-place piles and shafts, the nominal concrete strength shall not be greater than f'c = 4,000 psi

B. Reinforcing Steel

Reinforcing steel for concrete reinforcement including spiral reinforcement shall conform to ASTM A706/706M, Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.

Plain wire for welded wire fabric shall comply with ASTM A82, Specification for Steel Wire, Plain, for Concrete Reinforcement.

C. Concrete Cover

Minimum concrete cover shall conform to AASHTO LRFD with Caltrans Amendments Table 5.12.3-1, with the following exceptions:

- Uncased drilled shafts: 6 inches
- Cased drilled shafts with temporary casing: 4 inches

D. Prestressing Steel

Prestressing steel shall conform to the requirements of ASTM A416/A416M, or ASTM A722. Prestressing strand or wire shall be low relaxation. Additional requirements follow:

- Only post-tensioning systems that utilize tendons fully encapsulated with grout within the anchorages and ducts are allowed.
- Embedded anchors for bars are permitted.
- Strand or tendon couplers are not permitted.
- Select the post-tensioning grout for use by the proper application either repair, horizontal, or vertical. Grout will be mixed with potable water.
- Grout
  - Only pre-packaged grout mixes designed for the specific application are permitted. The grout shall not contain aluminum or other components which produce hydrogen, carbon dioxide or oxygen gas.
  - Chemical testing of a fresh dry sample taken from a bag in each lot of prepackaged grout shall be performed to determine chloride concentrations in accordance with the following requirement. Total chloride ions shall be less than 0.08% measured by weight of cementious material according to ASTM C1152.
- Anchorages
  - Ensure that anchorages develop at least 95 percent of the actual ultimate tensile strength of the prestressing steel when tested in an unbonded state, without exceeding the anticipated set.
  - Design anchorages so the average concrete bearing stress is in compliance with AASHTO "Load and Resistance Factor Design (LRFD) Bridge Design Specifications."
  - Test and provide written certification that anchorages meet or exceed the testing requirements in the AASHTO "LRFD Bridge Construction Specifications".
  - Equip anchorages with a permanent grout cap vented and bolted to the anchorage. Provide wedge plates with centering lugs or shoulders to facilitate alignment with the bearing plate. Cast anchorages with grout outlets suitable for inspection from either the top or front of the anchorage. The grout outlet shall serve a dual function of grout outlet and post-grouting inspection access. The geometry of the grout outlets must facilitate being drilled using a 3/8-inch-diameter straight bit to facilitate endoscope inspection directly behind the anchor plate. Anchorages may be fabricated to facilitate both inspection locations or may be two separate anchorages of the same type – each providing singular inspection entry locations.

- Anchorages shall be protected with epoxy grout encapsulation with an elastomeric coating

- Ducts and Pipes
  - Do not use ducts manufactured from recycled material.
  - Use seamless fabrication methods to manufacture ducts.
  - Ferrous metal ducts shall not be used.
  - Precast segmental bridges with internal tendons shall use segmental duct couplers with 6 degrees of alignment allowance at all segment joints.

E. Structural Steel

Structural steel design for bridge-type structures shall meet the requirements of the AASHTO LRFD with Caltrans Amendments.

Structural Steel Shapes shall conform to ASTM A6. Additional properties are as follows:

Wide flange shapes:	ASTM A992
M-shapes, S-shapes, HP shapes:	ASTM A572
Angles, Channels:	ASTM A572
Rectangular and square hollow sections:	ASTM A500 Gr B (46 ksi)
Round hollow sections:	ASTM A500 Gr B (42 ksi)
Steel pipe:	ASTM A53 Gr B (35 ksi)
Plates, Bars:	ASTM A36 (36 ksi)
Bolts:	ASTM A325
Nuts:	ASTM A563
Washers:	ASTM F436

Welding of built up members and steel fabrications shall comply with AASHTO/AWS D 1.5

Welding of HSS sections and pipes shall comply with AWS D 1.1

Miscellaneous steel items shall be hot-dip galvanized after fabrication unless completely embedded in concrete and unless noted otherwise

Splice Locations – If splicing of a structural steel member is permitted, indicate the location of the splice. Such locations shall be at or near a cross section of minimum stress.

12.8.2 Design Loads and Effects

Aerial structure loads and load combinations are specified in Section 12.5. Track-Structure Interaction requirements and those specific load combinations are specified in Section 12.6. Seismic requirements are specified in the *Seismic* chapter.

12.8.3 Foundations

12.8.3.1 Shallow Foundation Design

Shallow foundations such as spread footings shall be designed in accordance with AASHTO LRFD with Caltrans Amendments. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical reports described in the *Geotechnical* chapter. Use of presumptive values shall not be allowed.

12.8.3.2 Deep Foundation Design

Design of deep foundations shall be based on project-specific information developed for the location(s) and foundation type planned. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical reports described in the *Geotechnical* chapter. Use of presumptive values shall not be allowed. Bottom clean out of drilled shafts constructed using the wet method shall be verified.

Where permanent steel casing is used for structural capacity, it shall have a minimum wall thickness of 3/4 inch and be provided with internal shear lugs if composite action is to be relied upon. Additionally, the design basis of the steel section shall be reduced to account for corrosion over the life of the structure based on actual soil and ground water conditions. A site specific corrosion study shall be performed to determine the deduction of the wall thickness due to the corrosive characteristics. A minimum of 1/8 inch reduced wall thickness shall be applied. Steel casing shall not be considered for structural support in extremely aggressive environments.

Construction tolerance for drilled shafts shall be in accordance with AASHTO LRFD with Caltrans Amendments. For trackway shafts greater than 5 feet in diameter, the drilled shafts shall be designed assuming they are offset at the top of the shaft a minimum of 6 inches. Refer to the Standard Specification on Drilled Concrete Piers and Shafts.

Geotechnical Design of mini-piles shall be in accordance with AASHTO LRFD with Caltrans Amendments, Section 10.9 Micropiles and FHWA-SA-97-070 (Micropile Design and Construction Guidelines, June 2000).

The upper 5 feet as measured from lowest adjacent grade shall be discounted in any axial and lateral load analyses except where it can be shown that measures are provided to prevent future excavations around the pile within three diameters from the shaft or pile group exterior surface. For the analysis to obtain the demand forces the upper 5 feet shall be considered.

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12.8.4 Steel Structures

Steel through trusses and through girders may be used for longer spans requiring minimal structure depth and other steel built up sections, beams and girders may be used over railroads or highways. For through and semi-through type bridges the requirements of Section 12.5.2.13 shall be met for structural elements within 16 feet of the track centreline. Loadings shall be considered as derailment loads applied in the Extreme 1 Load Combination in Table 12-4. .

12.8.4.1 Continuous Steel Structures

For continuous girders and other statically indeterminate structures, the moments, shears, and thrusts produced by external loads shall be determined by elastic analysis. The effects of creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlements shall be considered in the design.

12.8.4.2 Fracture Critical Members

Fracture critical members shall be designed in accordance with AASHTO LFRD with Caltrans Amendments. A load factor of 1.50 shall apply to live load of the Fatigue load combination described in Table 12-4. Welding in tension zones in fracture critical members is not permitted.

12.8.4.3 High Performance Coating

Steel Bridges shall have a high performance coating system such as polysiloxane, polyaspartic modified urethane, or fluoropolymer which may be applied in the field. Primer shall be inorganic or organic zinc as recommended by the manufacturer of finish coats. Coatings including primers shall comply, at a minimum with South Coast Air Quality Management District (SCAQMD) Rule 113.

The Contractor shall provide services of an independent coating inspector. Independent coating inspector shall be certified under NACE International’s Certified Inspector Program as a Certified Coating Inspector.

12.8.4.4 Orthotropic Steel Decks

Steel orthotropic plate decks shall not be used for the HST mainline structures.

12.8.4.5 Bearing Replacement

Reinforced jacking points shall be identified clearly on As-Built drawings.

12.8.4.6 Inspection and Maintenance

Construction details of steel bridges shall facilitate maintenance and inspection so that inspection and maintenance can be performed during non-revenue service. Structures over railroads or highways shall provide for the inspection and maintenance during limited access to the below deck elements. Steel box girders and box beams shall have access hatches to allow maintenance and inspection of the member.

12.8.5 Concrete Structures

Aerial structures, bridges and grade separation superstructures may be constructed using cast-in-place concrete, precast girders either single span, span by span or segmental, as well as cast-in-place, segmental balanced cantilever or incrementally launched methods. Concrete through girders shall meet the same requirements as steel through girders.

The CEP-FIP Model Code for Concrete Structures shall be used for determining time dependent effects due to creep, shrinkage and prestressing steel relaxation.

12.8.5.1 Longitudinal Tension Stresses in Prestressed Members

AASHTO LRFD with Caltrans Amendments shall be used for allowable longitudinal tension stresses. Tension stresses are not allowed in pre-compressed tensile zones after all losses have occurred.

12.8.5.2 Additional Requirements for Segmental Trackway Construction

Shear and torsion design to conform to AASHTO LRFD, Article 5.8.6, in addition to AASHTO LRFD with Caltrans Amendments.

Principal tensile stresses in webs to conform to AASHTO LRFD with Caltrans Amendments, Article 5.8.5.

Precast segmental concrete construction with dry segment joints is not permitted. Joints in precast segmental bridges and viaducts shall be either cast-in-place closures or match cast epoxied joints.

Hollow columns shall have a solid section minimum 5 feet above finished grade or 12 feet above high water level. Vertical post-tensioning is not allowed in the solid sections. An access opening shall be provided near the top of the column. See Section 12.8.10 for sizes. Access openings shall be located outside of potential plastic hinge zones. Internal platforms and ladders shall be provided.

Two inch diameter vent holes are required through the bottom flange of box girders near each end of each span.

12.8.5.3 Crack Control

The design of prestressed concrete or reinforced concrete aerial structures shall consider the effect of temporary loads imposed by sequence of construction stages, forming, falsework, and construction equipment, as well as the stresses created by lifting or placing pre-cast members, stress concentration (non-uniform bearing at the ends of pre-cast beams), end block design and detailing, methods of erection, shrinkage, and curing. Ensure that the structural design and detailing of pre-stressed or reinforced concrete members is adequate and meets durability requirements and that specifications are prepared which are compatible with the design so that crack widths are no greater than allowed by AASHTO LRFD with Caltrans Amendments Class 2 exposure condition in construction stages or service. If the concrete member is continuously submerged in water or is a zone of intermittent wetting and drying the exposure factor used in AASHTO LRFD with Caltrans Amendments Article 5.7.3.4 shall be 0.25 or less.

12.8.5.4 Maintenance and Inspection of Concrete Structures

Inspection and maintenance hatches shall be provided into each closed girder. These hatches may be through the girder soffits or in combination with openings between adjacent girder diaphragms.

The minimum headroom inside of typical box girders shall be 6 feet. For two- or three-span short bridges with spans less than 90 feet, the minimum headroom inside of box girders shall be 4 feet.

Minimum clearance from girder ends to bearings and from abutment backwalls to bearings shall be 2.5 feet to allow for access.

In-span hinges and associated expansion joints are not allowed.

12.8.5.5 Continuous Concrete Structures

For continuous girders and other statically indeterminate structures, the moments, shears, and thrusts produced by external loads and prestressing shall be determined by elastic analysis. The effects of creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlements shall be considered in the design.

12.8.6 General Aerial Structure and Bridge Design Features

12.8.6.1 Bridge Skew

The preferred angle of crossing and bridge structure relative to the centerline of track is 90-degrees. In cases where a 90-degree crossing cannot be constructed, the skew of the bridge shall be limited so that for each track the deck end is between successive rail fasteners and the applicable provisions of Section 12.6 Track-Structure Interaction are met. The maximum skew of a bridge from 90-degrees shall not exceed 30-degrees.

12.8.6.2 Embankment Length Between Abutments

The length of embankment between abutments shall not be less than 500 feet. The length of embankment between an abutment and a culvert shall not be less than 100 feet. If closer spacing is required, then the embankment shall be specially treated such that a constant gradient of stiffness shall be provided between the two adjacent bridge deck stiffnesses. Refer to the *Geotechnical* chapter for specific requirements for embankment fills and abutment backfill.

12.8.6.3 Containment Barriers

Containment barriers as described in Section 12.5.2.13-B are required on all bridges and aerial structures.

12.8.6.4 Intrusion Protection

Aerial structures shall be protected from errant highway vehicles as well as from derailed trains as described in the *Rolling Stock and Vehicle Intrusion Protection* chapter and as required in the following:

A. Highway Traffic Intrusion

HST substructures, as required by the *Rolling Stock and Vehicle Intrusion Protection* chapter, shall be protected by an appropriate barrier as specified AASHTO LRFD with Caltrans Amendments Article 3.6.5.1 or designed for the force presented in AASHTO LRFD with Caltrans Amendments, Article 3.6.5.2.

B. Railroad Intrusion

HST substructures located adjacent to conventional railroad shall be protected as specified in the *Rolling Stock and Vehicle Intrusion Protection* chapter. The design shall follow the requirements of AASHTO LRFD with Caltrans Amendments, Article 2.3.3.4. If an independent intrusion barrier is not provided, the substructure shall be designed to resist forces presented in Section 12.5.2.14.

12.8.6.5 Uplift

There shall be no net uplift at any support for any combination of loadings.

12.8.6.6 Friction

Friction shall be considered in the design where applicable.

12.8.6.7 Sound Barriers

Sound barriers, both presence and absence, shall be considered in the evaluation of stress, vibration, and deflection limits.

12.8.6.8 Drainage

Drainage of water from aerial structures, bridges and grade separations shall be accomplished by sloping the deck towards the center of the deck, and sloping the girders towards a pier support or abutment. Water shall be collected and conveyed to a drainage pipe cast into the concrete substructure. The pipe shall pass through the pier columns and abutment walls to exit through the foundations. Column reinforcing in potential plastic hinge zones shall not be interrupted for drain pipes. Refer to the *Drainage* chapter for other requirements.

12.8.6.9 Expansion Joints

Expansion Joints shall be provided between girder ends and between girder end and abutment walls to allow superstructure movements and prevent water and other material from falling from the superstructure. Expansion joints are not required to resist highway or rail traffic loads, but shall be protected from ballast if ballast is used. Expansion joints may be part of a bridge drainage system.

The design life of expansion joints is given in the *General* chapter. Expansion joints shall be detailed to allow replacement during the five hour non revenue service maintenance work window. Refer to Section 12.6 for length of thermal units.

Rail expansion joints are described in the *Trackwork* chapter with uses and limitations.

A. Structural Expansion Joints (SEJ)

The detailed design of structural expansion joints shall:

1. Provide free movement space in the bridge longitudinal direction for:
   - creep, shrinkage, temperature variation, braking and acceleration.
   - creep, shrinkage, temperature variation, single track braking and OBE.

   Expansion joints and connections to the structure shall be capable of resisting loads transmitted through the ballast under these loading conditions.
2. Provide enough space between two adjacent structures to prevent unbuffered impact between them during a MCE earthquake.
3. For ballasted track, if buried type expansion joints are used, no part of the structure expansion joints shall protrude above top surface of the protection layer for waterproofing membrane.

The Contractor shall adjust the joint gap during installation to accommodate the effects of prestressing including shrinkage and creep, and the difference between the ambient temperature and the design temperature.

An assessment of the longitudinal actions shall be made and the movement checked in accordance with the requirements of Section 12.6.

The installation of turnouts and SEJ shall be based on requirements in Section 12.8.6.9-B.

B. Structure Expansion Joints in Special Trackwork

Structural joints may be placed within turnout and crossover units as needed to minimize relative movement between structures and track. Structural joints shall not be located within areas of special track supporting plates nor within the vicinity of the movable portions of switches and frogs. Longitudinal joints in structures shall not be located under crossover tracks. Structural joints under special trackwork units shall be perpendicular or close to perpendicular to the orientation of the track. Potential movement of the structure relative to the track shall be oriented with the alignment of the track.

Permissible and prohibited locations for joints are illustrated in Figures 12-34 and 12-35 and the limiting location dimensions given in the Tables 12-25 and 12-26.

Figure 12-34: Joint Location Limitations at Low-Speed Turnouts

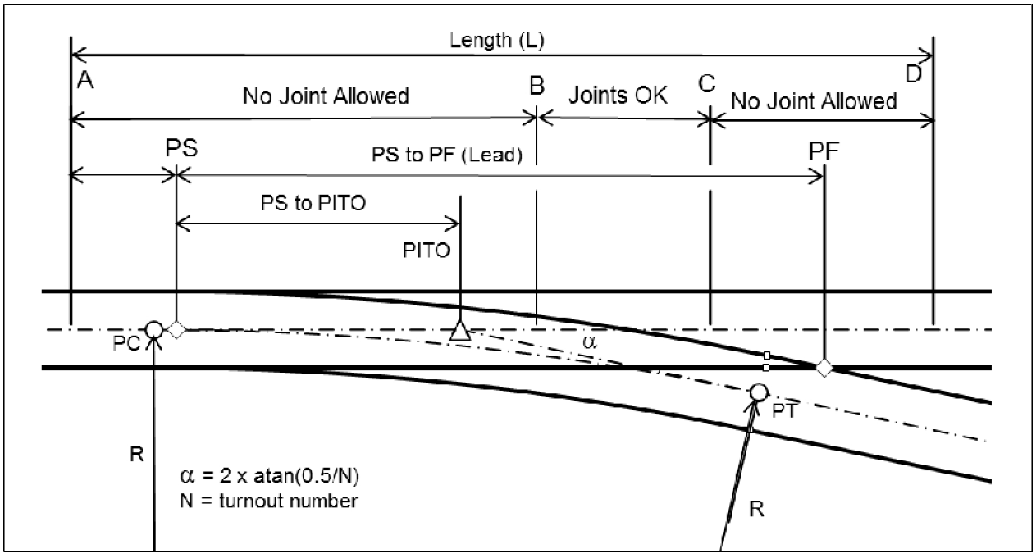

Table 12-25: Joint Location Limitations at Low-Speed Turnouts

Turnout Properties					Location of points defining limits of joints (feet)				Length of No Joint / Allowed Joint Zones (feet)		
Internal Radius (feet)	PC to PS (feet)	N	Turnout angle α	PS to PI (feet)	PS to A	PS to B	PS to C	PS to D	No Jt. A to B	Jt. OK B to C	No Jt. C to D
620.0	1.69	9	6d21m35s	32.75	20.0	40.6	62.2	86.8	60.6	21.5	24.6
950.0	1.93	11	5d12m18s	41.25	20.0	50.3	77.0	107.2	70.3	26.7	30.2
1,750.0	2.58	15	3d49m06s	55.75	25.0	68.3	104.5	145.8	93.3	36.2	41.3
3,275.0	3.25	20	2d51m51s	78.00	25.0	93.1	142.5	198.0	118.1	49.4	55.5
4,650.0	3.87	24	2d23m13s	93.00	25.0	111.3	170.4	237.0	136.3	59.1	66.6

Figure 12-35: Joint Location Limitations at High-Speed Turnouts

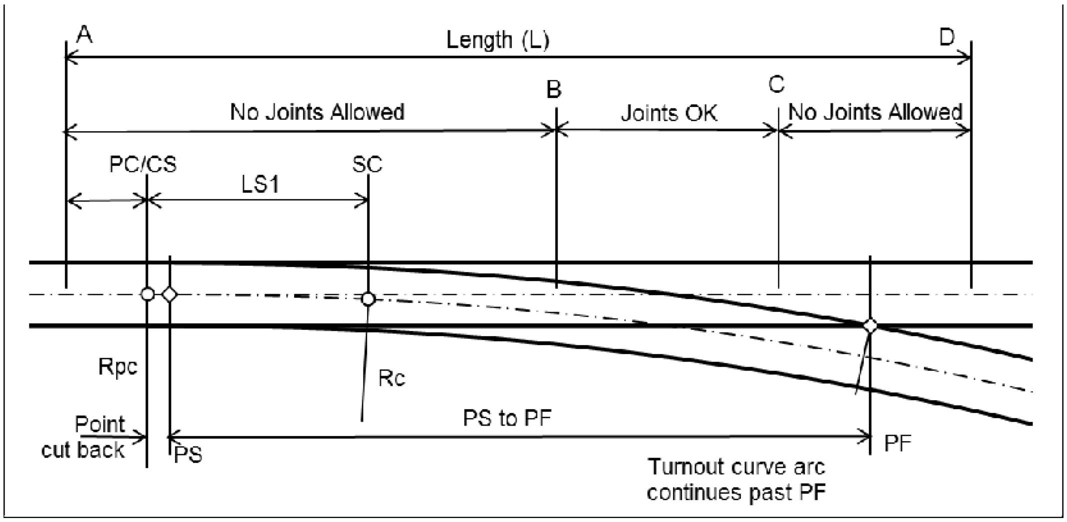

Table 12-26: Joint Location Limitations at High-Speed Turnouts

Turnout Properties					Location of points defining limits of joints (feet)				Length of No Joint / Allowed Joint Zones (feet)		
Design Speed (mph)	Entry Radius Rpc (feet)	entry spiral length (feet)	body radius Rc (feet)	Switch point cutback (feet)	PC to A	PC to B	PC to C	PC to D	No Jt. A to B	Jt. OK B to C	No Jt. C to D
60	10,000	90.00	5,000	6.00	24.0	134.2	207.3	270.7	158.2	73.0	63.4
80	18,000	120.00	9,000	8.00	22.0	180.0	278.0	363.1	202.0	98.0	85.1
110	34,000	160.00	17,000	11.00	29.0	246.6	381.1	498.0	275.6	134.5	117.0
150	80,000	220.00	32,000	16.00	24.0	348.9	533.6	694.2	372.9	184.8	160.6

For the 150 mph turnout a design variance will be required for structural arrangement.

12.8.6.10 Longitudinal joints in Special Trackwork

In zones of special trackwork, where tracks will cross between parallel superstructure elements such as girders, those superstructure elements shall be connected into a continuous deck that can support the tracks as well as a derailed train with loads described in Section 12.5.2.13. The deck shall be cast in place or made continuous with a longitudinal closure strip between decks. The transverse strengthening shall also include rigid diaphragms, post-tensioning, welded steel plates or other such strengthening elements. Railroad box sections post-tensioned together shall be considered to have a continuous deck. This continuity shall extend from the Point A indicated in Figures 12-34 and 12-35 before the point of switch to the equivalent Point A at the opposite end of the turnout beyond the final point of tangent in the special trackwork. This continuity is independent of the structural expansions describes in Section 12.8.6.9.

12.8.6.11 Bearings

AASHTO LRFD with Caltrans Amendments shall be used for design of bearings. Elastomeric bearings, disk bearings, spherical bearings and seismic isolation bearings are allowed. If seismic isolation bearings are used, the design shall follow the requirements of the *Seismic* chapter. Longitudinal and lateral restraints shall be placed to minimize eccentric deformations.

The design life of bearings shall be as presented in the *General* chapter. Since bearings will be replaced during the life of the structures an inspection and replacement plan for bearings shall be provided. Inspection and replacement will be allowed only during the non-revenue service hours of the HST such that train operations are not affected by the inspection or replacement.

12.8.6.12 Rail Stresses and Deformations of Aerial Structures

High-speed trains are very sensitive to deformations of the tracks. See Section 12.6 – Track-Structure Interaction for additional requirements related to track structure interaction, resonance and dynamic performance.

12.8.6.13 Resonance of High Speed Trains on Repetitious Spans

Amplification of vibrations has been observed on high-speed trains traveling on long viaducts where the same span is repeated many times. It affects ride quality. In order to limit this response long viaducts shall have span length modified every 20 spans or 2000 feet. The modification shall include at least two spans reduced in length by 20 percent from the typical span between column centers. Following spans shall resume from the new column location rather than from the original column layout.

12.8.6.14 Camber and Deflections for Aerial Trackway Structures

Structures shall be built with camber equal to the sum of deflections under dead load of steel structural components and permanent attachments (DC), dead load of non-structural and non-permanent attachments (DW), and one track of modified Cooper E-50 loading plus impact (LLRM+I)[1].

As a guide in design, the total long-term predicted camber growth, less deflection due to full dead load, shall be less than 1/5000 of the span length for prestressed concrete aerial structures measured 10,000 days after casting concrete.

To ensure rider comfort, the deflection of longitudinal girders under normal live load plus dynamic load allowance shall be as described in Section 12.6 – Track-Structure Interaction.

12.8.6.15 Structure Deformation and Settlement

The control of deformations through proper structural design is of paramount importance in obtaining acceptable ride quality for the rail vehicles and passengers. Consider structure deformations, including foundation settlement, for their effects on structural behavior but also and on trackwork. As a minimum, trackway piers and abutments settlement as measured at the top of concrete of the finished trackway girder deck shall be limited as prescribed in the *Geotechnical* chapter.

12.8.6.16 Superelevation on Aerial Structures

Superelevation of tracks through curved track shall be accomplished through the trackwork. Girder decks shall maintain a level attitude transverse to the track with deck slope allowed only for drainage.

12.8.6.17 Walkways, Parapets, and Sound Walls

Loads on walkways shall be as described in Section 12.7.1.4. Walkways shall be precast or cast-in-place concrete. The walkway shall be on the cover of the cable trough. The cable trough cover shall be a non-skid material and the cover shall be anchored using positive connections to resist loads described Section 12.5. Walkways shall follow requirements on the Standard and Directive Drawings.

Parapets shall be provided along edges of aerial structures, bridges, and HST grade separation structures. Parapets shall be designed for wind loads, slipstream effects, and other loadings in Section 12.5. Parapets shall be designed to accommodate installation of sound walls.

Parapets and safety railings shall be designed to withstand the forces in Section 12.7.1.6. In locations where conduit risers are required along the alignment, parapets may be required to support conduits.

Temporary railings may be necessary to provide safety after girders are placed and before parapets are placed. Temporary railings shall meet the same requirements as safety railings. Temporary railings may be needed between construction contracts.

The height of sound walls shall be determined based on results from the noise attenuation study. The sound wall and its connection to the structure shall be capable of resisting the slipstream effects from passing trains and the wind load as described in Section 12.5. No gap shall be permitted between the bottom of sound wall and the structure deck, nor any vertical gaps between the sound wall panels.

12.8.7 Complex and Non-Standard Aerial Structures

For the definitions of standard, complex and non-standard structures see *Seismic* chapter. Some structures and structural systems involve unique design, construction, and performance problems not covered by these criteria. Some specific requirements follow:

- Straddle and outrigger bents have cap beams that extend beyond the edges of superstructure toward columns located outside of the superstructure.
  - The load path necessary to accommodate longitudinal actions of the superstructure shall be defined in a report and submitted with the Type Selection Report to the Authority, see Section 12.8.1.
  - Torsion cracking in the primary load path is not permitted in concrete beam members. Compatibility torsion is allowed.

– Torsional rotation of concrete columns is not permitted under seismic actions when high bending and shear stresses occur.

12.8.8 Emergency Access

Where access stairs are provided, the emergency stairs shall be at the edge of deck. A safety gate opening away from the track shall be provided at each access opening. The safety gate shall be designed to resist wind and slipstream forces from Section 12.5 as well as live loads in Section 12.7.

12.8.9 OCS Pole and Traction Power Facility Gantry Supports

Girder decks shall be designed to accommodate and support OCS poles and Traction Power Facility Gantry. Loads are described in Section 12.5. Conduit or sleeves for future conduit shall be provided from external power sources to the OCS poles and Traction Power Facility Gantry.

12.8.10 Maintenance of HST Aerial Structures, Bridges, and Grade Separations

Because of the large number of structures along the line, special care shall be taken in the design to reduce maintenance requirements. The following requirements shall apply:

- Reinforced or prestressed concrete structures are preferred over steel structures. The Designer shall justify in writing the use of steel structures, demonstrating the benefits of any steel structure.

- Bearings shall be easily accessible for inspection. They shall be adjustable and replaceable at any time during the life of the structure without disrupting train normal operations. Bearing replacement shall be completed within non-revenue hours. During this period train speed may be limited at location where bearing is being replaced. The design documents shall provide a description of the procedures for bearing replacement, including the location of the jacks with safety nuts and a calculation of forces.

- Access arrangements for maintaining exterior surfaces or equipment attached thereto shall be provided from the ground or from movable gantries. At intervals not greater than 300 feet and at one location in the case of a shorter isolated structure, 2.5 feet x 5 feet access openings with steel grating for inspection and maintenance shall be provided in the bottom slabs close to the expansion joint piers. For tall and long structure access openings may be at both ends near abutments.

- If a pier is not accessible from the ground beneath, such as river crossing bridge, a 3 feet x 3 feet access opening with steel grating shall be provided in the bottom slab of decks so that the pier top can be reached from the inside of the box girder. Beside this opening, a work platform shall be provided at the pier top for the maintenance of bearings.

- In the design of bridges, the vertical load of maintenance the gantry on the deck overhang shall be taken into consideration. The maintenance gantry shall be represented by a line load of 30 kips over a length of 13 feet applied to the edge of the cantilever deck.
- A lifting hook capable of lifting 7500 pounds shall be embedded in the underside of the superstructure top slab above each access opening. Access openings shall be equipped with lockable galvanized steel hatches.

12.9 Requirements for Bridges not Supporting HST

Bridges not supporting HST are classified as either Primary Type 2 or Secondary in the *Seismic* chapter. Secondary Bridges shall be designed to either the requirements of AASHTO LRFD with Caltrans Amendments for pedestrian and highway bridges, or AREMA Manual for Railway Engineering for railroad bridges. Primary Type 2 structures shall meet the requirements described below.

12.9.1 Load Requirements for New Primary Type 2 Pedestrian Bridges

New Primary Type 2 pedestrian bridges shall be designed according to AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, with seismic design according to the *Seismic* chapter.

New Primary Type 2 pedestrian bridges shall not have expansion joints or other features requiring special maintenance in the HST right of way. Access shall be reserved for inspection and routine maintenance without impacting HST operations or endangering bridge maintenance inspectors.

Local County, City, or third party bridges shall follow requirements of the local jurisdiction.

12.9.2 Load Requirements for New Secondary Pedestrian Bridges

Bridges or structures that support pedestrian loadings not spanning and classified as Secondary structures shall be designed according to AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges and with the seismic provisions of CSDC. OBE is not required unless seismic performance has the potential to directly impact HST service.

12.9.2.1 Existing Pedestrian Bridges

Existing Primary Type 2 pedestrian bridges shall be evaluated to determine a Bridge Health Index. HST horizontal and vertical clearances shall meet the clearance requirements specified in the *Trackway Clearances* chapter. Existing Primary Type 2 pedestrian bridges that are load restricted or a Bridge Health Index (Sufficiency Rating) less than 80 shall be replaced.

For existing Primary Type 2 pedestrian bridges that do not meet seismic performance requirements according to the *Seismic* chapter, retrofit shall be required and a Seismic Design and Analysis Plan submitted as required in the *Seismic* chapter.

12.9.2.2 Live Load or Pedestrian Bridges

Areas where cash carts are used shall be designed to accommodate a point live load of 350 pounds per wheel.

Areas where station equipment will be transported shall be designed for a load of 2000 pounds spread over a square with sides of two feet.

12.9.2.3 Frequency and Vibration Limits for Pedestrian Bridges

Pedestrian bridges or structures shall meet the requirements for pedestrian comfort specified in Section 12.7.1.13 – Frequency and Vibration Limits.

12.9.3 Requirements for Highway Bridges

Bridges or structures that support highway loadings not spanning over HST structures or alignments, shall be designed according to AASHTO LRFD with Caltrans Amendments and with the seismic provisions of CSDC and follow the Office of Specially Projects Information and Procedures Guide for Planning Studies and Type Selection to achieve approval from the Department of Transportation. County, city and other bridges shall follow requirements of these jurisdictions.

12.9.3.1 New Highway Bridges

New highway bridges that are classified as Primary Type 2 structures shall be designed according to AASHTO LRFD with Caltrans Amendments and with seismic design following the requirements of the *Seismic* chapter and meeting the durability requirements herein in addition to requirements of the bridge owner.

New Primary Type 2 highway bridges shall not have expansion joints or other features within the HST right of way requiring maintenance. Access shall be reserved for safe inspection and normal maintenance of the highway bridge over the HST without impacting train operations or endangering bridge maintenance inspectors.

Bridges supporting the California State Highway System (SHS) shall follow the procedures in the State of California Department of Transportation procedures described in the Office of Specially Projects Information and Procedures Guide for Planning Studies and Type Selection to achieve approval from the Department of Transportation. County, city and other bridges shall follow requirements of these jurisdictions in addition to the requirements herein.

12.9.3.2 Existing Highway Bridges

In situations where HST must pass under an existing highway bridge and are thus classified as Primary Type 2 bridges, the highway bridge shall be evaluated for its continued service. Each bridge shall:

- be accepted as-is
- or, repaired or retrofitted to meet these criteria

- or replaced with a new bridge

The determination of the appropriate selection starts with the current Caltrans Bridge Health Index to be provided by Caltrans. Structures that are load restricted or a Bridge Health Index (Sufficiency Rating) equivalent to or less than 80 shall need replacement. Bridge Health Index less than 90 shall require repair to bring the Index to above 98 prior to start of train service.

The seismic performance requirements described in the *Seismic* chapter for Primary Type 2 bridges shall be satisfied. For existing SHS structures that do not meet HST structural or seismic performance requirements as described in the *Seismic* chapter, an As-Built assessment shall be performed based upon best available information and in accordance with HST for Primary Type 2 bridges and described in the Seismic Design and Analysis Plan required in the *Seismic* chapter. For structures not meeting these requirements a seismic retrofit shall be required. The proposed seismic retrofit shall be prepared according to the Seismic Design and Analysis Plan and reviewed by Caltrans and the Authority for agreement on a complete seismic retrofit strategy for both MCE and OBE risk levels and the corresponding performance described in the *Seismic* chapter.

The horizontal and vertical clearances shall be evaluated by the Contractor to meet the requirements in the *Trackway Clearances* chapter. Structures that cannot meet clearance requirements or cannot obtain variance approval shall require modification or replacement.

Evaluation will consider if the bridge is structurally deficient or functionally obsolete. Continued operation of the "as is" facility, seismic retrofit/rehabilitation, or replacement will be made by the Authority in conjunction with the owner and the Contractor.

Mitigation measures shall be considered for all existing structures. Such mitigations may include modifying HST alignments to satisfy clearance envelopes; constructing track protection shields to keep structural debris from affecting train service; constructing catcher frames to support displaced existing girders to minimize impacts to HST due to post-seismic event recovery; and/or replacement of the existing structure.

12.9.4 Requirements for New Railway Bridges

New Primary Type 2 railway bridges shall be designed according to AREMA and the requirements of the railroad owner and operator with seismic design according to the *Seismic* chapter.

New Primary Type 2 railway bridges shall not have expansion joints or other features requiring special maintenance in the HST right of way. Access shall be reserved for inspection and routine maintenance without impacting HST operations or endangering bridge maintenance inspectors.

Existing freight rail bridges spanning over the HST right of way shall be inspected, assessed and evaluated to prepare a recommendation for either acceptance "as is", seismic retrofit/repair, or replacement by the Contractor. The seismic requirements for a primary structure as defined in the *Seismic* chapter shall be followed.

12.9.5 Requirements for Existing Railway Bridges

Existing Primary Type 2 railway bridges shall be evaluated to determine a Bridge Health Index. A Bridge Health Index of less than 80 shall require a bridge replacement. HST horizontal and vertical clearances shall meet the clearance requirements specified in the *Trackway Clearances* chapter.

For existing Primary Type 2 railway bridges that do not meet seismic performance requirements according to the *Seismic* chapter, retrofit shall be required and a Seismic Design and Analysis Plan submitted as required in the *Seismic* chapter.

12.9.6 Intrusion Protection

If a pedestrian, highway, or railway bridge column is located adjacent to the HST alignment, then intrusion protection shall be provided. Barriers shall be designed to meet the requirements in Sections 12.5.2.14 and 12.7.1.9. Intrusion protection requirements are defined in the *Rolling Stock and Vehicle Intrusion Protection* chapter.

12.10 Earth Retaining Structures

Earth retaining structures shall be designed to lateral earth pressures in accordance with the *Geotechnical* chapter. Top of retaining walls, including fill, cut, and trench walls, shall be at least 1 foot above finish grade and provided with fall protection barriers as per Cal/OSHA. Wall heights may be higher as required for flood elevation and intrusion protection requirements. Walls with Access Deterring (AD) or Access Restricting (AR) fencing, per the *Civil* chapter, can serve as fall protection provided that the fencing meets Cal/OSHA requirements.

Temporary support excavation systems shall not be part of the permanent earth retaining structures.

12.10.1 Earth Pressure Acting on Retaining Structures

For earth pressure acting on retaining structures, see the *Geotechnical* chapter.

12.10.2 Retaining Walls

Retaining walls shall be designed in accordance with requirements of the *Geotechnical* chapter. Retaining walls shall be constructed with expansion joints in walls a maximum of 72 feet apart. Construction joints shall be a maximum of 24 feet apart.

12.10.3 MSE Walls

Mechanically stabilized earth walls shall be designed in accordance with requirements of the Geotechnical Reports in the *Geotechnical* chapter. If not specified in any reports, the MSE wall design shall follow the requirements specified in AASHTO LRFD section 11.10 with Caltrans amendments.

12.10.4 Trenches

Trenches are below grade structures with a retaining structure on both sides. Often the retaining structures are joined by a common reinforced concrete foundation. Waterproofing of the bottom of slab, and outside of walls is required if the top of concrete foundation slab is below the water table, see Section 12.11.3. For hydrostatic pressure (buoyancy), refer to Section 12.11.2.7.

Trench walls are considered to be rigid, so the minimum earth pressure coefficient is the At-rest earth pressure. Wall heights shall be based on flood and intrusion protection. The wall height shall not be less than 1 foot above finished grade.

12.10.5 Trench Intrusion Protection

HST trench structures shall be protected from errant highway vehicles and derailed trains as described in the *Rolling Stock and Vehicle Intrusion Protection* chapter and as required in the following.

12.10.5.1 Highway Traffic Intrusion

HST trench structures shall be protected by a continuous Caltrans type concrete barrier as specified in the *Rolling Stock and Vehicle Intrusion Protection* chapter. The trench wall shall be designed for the force presented in AASHTO LRFD with Caltrans Amendments Article 3.6.5.

12.10.5.2 Railroad Intrusion

HST trench structures located adjacent to conventional railroad shall be protected as specified in the *Rolling Stock and Vehicle Intrusion Protection* chapter. Where an independent intrusion protection cannot be constructed due to limited space, the trench wall, next to the conventional railroad, shall be constructed as described in *Rolling Stock and Vehicle Intrusion Protection* chapter and the wall shall be designed to resist forces presented in Section 12.5.2.14.

12.10.6 Struts

Struts may be used to support earth pressures in trenches. The minimum height of struts shall be 27 feet clear from TOR.

12.10.7 Trench Drainage

Trenches shall be drained to the low point of sag curves. Sump pumps and an interconnected sump and pump room shall house the pumps. Earth pressures on the sump structure shall be as required in the Geotechnical reports described in the *Geotechnical* chapter. Sump structures shall be made water proof from ground water. Refer to the *Drainage* chapter.

12.10.8 Trench Emergency Exits

Emergency exits are required from trenches at a minimum spacing of 2,500 feet. There shall be at least one emergency exit in each trench. The exit shall include an enclosed stairway from the walkway to ground surface and a secured head house at the surface.

12.11 Cut-and-Cover Structures

The criteria set forth in this section govern the static load design of cut-and-cover underground structures with the exception of pile foundations. Cut-and-cover structures include line structures, crosspassages, sump pump structures, underground stations, vaults, ventilation structures, and other structures of similar nature. Portal and ventilation requirements and minimum cross sectional tunnel areas shall be as required in the *Tunnels* chapter.

The design of structures within the scope of this section shall be in accordance with the provisions set forth in these criteria and shall also meet the requirements of the AASHTO LRFD with Caltrans Amendment, CBC, ACI, AISC and AWS, except where such requirements are in conflict with these criteria.

12.11.1 Structural System

Structural system for cut-and-cover line structures shall be single and/or multi-cell reinforced concrete box structures, with walls and slabs acting one-way in the transverse direction to form a continuous frame. Temporary excavation support systems shall not be used as whole or part of the permanent walls. Expansion or contraction joints are required at locations of major change in structural sections such as from line structure to station. Construction joints shall have continuous reinforcing steel, non-metallic waterstops and sealants.

12.11.2 Loads and Forces

Components of underground structures shall be proportioned to withstand the applicable loads and forces described in Section 12.5.

Cut-and-cover structures shall, at minimum, be designed for the forces described herein.

12.11.2.1 Zone of Influence

Zone of Influence is defined as the area above a positive Line of Influence which is a line from the critical point of substructure at a slope of 2 horizontal to positive 1 vertical (line sloping towards ground level) or the area below a negative Line of Influence which is a line from the critical point of substructure at a slope of 2 horizontal to negative 1 vertical (line sloping away from ground level).

12.11.2.2 Future Traffic Loads

An area surcharge applied at the ground surface both over and adjacent to underground structures is to simulate possible roadway and sidewalk live loads. This surcharge is intended to simulate conditions during future construction activities adjacent to the underground structures. Such construction may result in permanent loads or in temporary loads from construction equipment from the stockpiling of construction materials, or from the deposition of excavated earth. It is possible that loads such as those from hauling trucks, may be applied inadvertently to the underground structures due to their innate inconspicuousness.

12.11.2.3 Alternative Traffic Loading

For the underground structures beneath or adjacent to operating railroads, both the vertical and lateral surcharge shall be based on Cooper's E-80, defined by AREMA MRE, railroad surcharge loadings. Refer to the standards of the subject railway.

For the underground structures adjacent to existing highway bridge overcrossings, both the vertical and lateral surcharge shall be based on the operating loads from the contractor's equipment with a minimum surcharge loadings equivalent to a 100-ton crawler crane.

For underground structures beneath highways, city streets or planned roadways, the applied vehicular live load shall be based on the HL-93 loading according to the AASHTO LRFD with Caltrans Amendments. For underground structures which are not anticipated to be beneath railroads, overcrossings, highways, streets, or roadways, the applied live load shall be based on no less than HL-93 loading according to the AASHTO LRFD with Caltrans Amendments. The distribution of this live load shall be in accordance with the following:

- Fill height less than two feet - live load shall be applied as concentrated loads directly to the top of the slab.
- Fill height greater than two feet - concentrated live loads shall be distributed over a square area, the sides of which shall equal 1.75 times the depth of the fill.
- When distribution areas overlap, the total load shall be uniformly distributed over an area defined by the outside limits of the individual areas.

For design of the top slab of underground structures supporting the alternative traffic loading, impact loading (I) shall conform to AASHTO LRFD with Caltrans Amendments, Article 3.6.2.2. The fill height shall be measured from the top of ground or pavement to the top of the underground structure.

12.11.2.4 Existing Structures

Existing structures that are to remain in place above underground structures shall either be underpinned in such a manner as to avoid increased load on the underground section, or the section shall be designed to support the structure directly. Third party structures shall be supported directly on HST structures only with specific approval in writing by the Authority.

Underground structures shall be designed for additional loading from existing adjacent buildings or structures unless they are permanently underpinned or have foundations to below the zone of influence. A building shall be considered to be adjacent to an underground structure when the horizontal distance from the building line to the nearest face of the underground structure is less than 2 times the depth of the underground structure invert below the building foundation.

Each existing structure shall be considered individually. In the absence of specific data for a given height of building and type of occupancy, applicable foundation loads shall be computed according to the CBC and the additional uniform lateral pressure on that portion of the underground structure sidewall below the elevation of the building foundation shall be distributed as shown on the preliminary engineering drawings. If distribution is not indicated on the preliminary engineering drawings, designer shall determine distribution.

12.11.2.5 Requirements for Future Structures

Some cut-and-cover structures must be designed to accommodate future structures in close proximity. Guidelines are provided below for a general case. Additional requirements may be required on a case-by-case basis.

A. Clearance

Structures over or adjacent to HST underground structures shall be designed and constructed so as not to impose any temporary or permanent adverse effects on underground structures. The minimum clearance between any part of the adjacent structures to exterior face of substructures shall be 7 feet–6 inches. Minimum cover of 8 feet shall be maintained wherever possible.

B. Surcharge

In general, cut-and-cover structures are designed with an area surcharge applied at the ground surface both over and adjacent to the structures. The area surcharge is considered static uniform load with the following value:

D (feet)	Additional Average Vertical Loading (psf)
D>20	0
5<D<20	800-40D
D<5	600

Where D is the vertical distance from the top of the underground structure roof to the ground surface.

C. Shoring

Shoring is required for excavations in the Zone of Influence. Zone of Influence is defined in Section 12.11.2.1.

D. <u>At-Rest Soil Condition</u>

See the *Geotechnical* chapter for soil loads and pressures needed for design.

E. <u>Soil Redistribution</u>

See the *Geotechnical* chapter for soil redistribution caused by temporary shoring or permanent foundation system.

F. <u>Dewatering</u>

Dewatering shall be monitored for changes in groundwater level. Recharging will be required if existing groundwater level is expected to drop more than 2 feet.

G. <u>Piles Predrilled</u>

Piles shall be predrilled to a minimum of 10 feet below the Line of Influence. Piles shall be driven in a sequence away from HST structures. No pile will be allowed between steel-lined tunnels.

H. <u>Vibration During Pile Driving</u>

Underground structures shall be monitored for vibration during pile driving operations for piles within 100 feet of the structures. Tunnels shall also be monitored for movement and deformation. Requirements for monitoring will be provided upon request.

I. <u>Future Excavation Adjacent to Cut-and-Cover</u>

The design of cut-and-cover structures shall consider an unbalanced lateral load condition due to possible future excavation or scour of 30 percent of total depth on one side of the structure. The designer shall, as a minimum, demonstrate by analysis that under this condition the structures will remain stable with an adequate margin of safety.

12.11.2.6 Earth Pressure

A. <u>Vertical Earth Pressure</u>

Depth of cover shall be measured from the ground surface or roadway crown, or from the street grade, whichever is higher, to the top of underground structure surface. Saturated densities of soils shall be used to determine the vertical earth pressure. Recommended values shall be presented in the Geotechnical reports described in the *Geotechnical* chapter.

B. <u>Lateral Earth Pressure</u>

For the purpose of these criteria, cut-and-cover box sections are defined as structures with stiff walls, which are restrained at the top so that the amount of deflection required to develop active pressure is not possible. See the *Geotechnical* chapter for earth pressures required for design.

12.11.2.7 Hydrostatic Pressure (Buoyancy)

The effects of hydrostatic uplift pressure shall be considered whenever ground water is present. The hydrostatic uplift pressure is a function of the height of water table above the foundation plane, and shall be assumed uniformly distributed across the width of the foundation in proportion to the depth of the base slab below the design ground water table.

Structures shall be checked for both with and without buoyancy to determine the governing design condition. Maximum design flood levels are indicated in the Hydrology Report. If the Hydrology Report is not part of the preliminary engineering documents provided by the Authority, the Designer (or its Geotechnical Engineer) shall determine applicable levels.

12.11.2.8 Flotation

For design flood levels and flood zone, see the Hydrology Report, if applicable.

Cut-and-cover structures subject to ground water table and/or located within the flood zone shall be checked and provided with adequate resistance to flotation.

No permanent dewatering system shall be assumed for the design of underground cut-and-cover structures.

A. <u>Factor of Safety</u>

The structure shall have a minimum factor of safety against flotation at any construction stage of 1.05, excluding any benefit from skin friction from perimeter of the structures.

The structure, when complete, shall have a minimum factor of safety against flotation at up to the 100-year flood level of 1.10 excluding skin frictional from perimeter of the structures.

The use of tiedowns, tension piles or other elements specifically designed to resist uplift forces shall be permitted and included in the flotation calculations. See *Geotechnical* chapter for other requirements buoyancy resisting elements.

The dead weight of the structure used in the flotation calculations for the underground structures shall exclude the weight of:

- Any building above the structure,
- Any live load internal or external to the structure,
- Any loads which is not be effective at the time, and
- 2 feet of backfill over the roof except when checking against the 100-year and 500-year flood levels.

12.11.2.9 Miscellaneous Loads

A. <u>Cut-and-Cover Walkway Cover Live Loads</u>

Stationary and hinged cover assemblies shall be designed for the loads on walkways per Section 12.7.1.4. Deflection at center of span under 100 pounds per square foot uniform live load shall not be more than 1/8 inch. Hinged cover material shall comply with NFPA 130 requirements.

B. <u>Live Loads and Equipment Loads for Ventilation Structures</u>

See Section 12.7 – Structural Design of Surface Facilities and Buildings, for roof and floor live loads and equipment loads for ventilation structures.

12.11.2.10 Seismic Design of Underground Structures

See the *Seismic* chapter for the demand requirements for seismic design of tunnels and underground structures. If ductility is required to meet seismic demands in underground structure, then the requirements provided in CSDC for lateral confinement reinforcing of concrete pier walls shall be satisfied.

12.11.2.11 Reinforced Concrete Box Station Sections

Underground station structures and their appurtenant structural elements such as entrances shall be designed in accordance with AASHTO LRFD with Caltrans Amendments referenced AASHTO specifications.

Subsurface exploration shall be carried out to determine the presence and influence of geologic and environmental conditions that may affect the performance of station structures and reported by one or more Geotechnical reports described in the *Geotechnical* chapter.

- Load combinations and load factors to be used are those provided in this chapter. Load resistant factors to be used are those provided by AASHTO LRFD with Caltrans Amendments and their referenced AASHTO Tables 3.4.1-2, 3.4.1-3, and 12.5.5-1. In addition, the effects of EH, EV, ES, LS, DD, DW, and WA shall be applied simultaneously in all their maximum and minimum values to produce the envelope of moment, torsion, shear, and axial force to produce the greatest demands to the structural framing. These load values shall cover the forces on the station structure at all phases of construction. See AASHTO LRFD with Caltrans Amendments Section 5.14.2.3.
  - Final ground induced pressures and design assumptions for soil-structure interaction shall be provided by the Geotechnical reports described in the *Geotechnical* chapter.
- Vertical pressure on foundation slabs may be divided into hydrostatic and earth pressure components. The hydrostatic component shall be distributed across the width of the foundation in proportion to the depth of each portion of the basic slab below the design groundwater table.
  - Distribution of the earth pressure moment shall be based on specified construction procedures, and will include elastic and plastic subgrade reaction foundation effects.
- For design, the horizontal earth pressure distribution diagram for multiple braced flexible walls shall be the trapezoidal pressure diagram as given by the Contractor's Geotechnical Engineer. Compression forces shall not be considered in shear design of the top and bottom slab in box sections.

12.11.2.12 Reinforced Concrete

Concrete for cut-and-cover structures shall be designed to attain the required chemical resistance to the environment, low permeability, water tightness and water absorption as specified in accordance with the Durability Report.

Concrete mixes shall be tested for acceptance by the selected Quality Assurance organization in accordance with the Quality Assurance Plan procedures to conform to the requirements.

Concrete for cut-and-cover structures shall also meet the requirements of the Standard Specifications, and the following minimum requirements:

- Strength – Minimum f'c shall be 4000 psi at 28 days.
- Proportioning Materials – The maximum water-cement ratio shall be 0.40 with 4.5 percent to 7.5 percent air entrainment.

12.11.2.13 Reinforcing Steel

Reinforcing steel in structural components shall use Customary U.S. Units, meet the requirements of the Standard Specifications, and meet the following requirements:

- Use reinforcing steel conforming to ASTM designation A 706 Grade 60 (F_y=60 ksi).
- Use uncoated reinforcing steel and welded wire fabric when the concrete surface is not in contact with soil/water (or waterproofing).
- Use epoxy coated reinforcing steel meeting the requirements of the Standard Specifications for all permanent concrete members when the concrete surface is in contact with soil/water (or waterproofing).
- Spacing of main reinforcement shall not exceed 12 inches.

12.11.2.14 Camber

The tunnel roof shall be cambered to mitigate the effect of long term loads (i.e., slab plus backfill). The camber shall be calculated in accordance with the AASHTO LRFD with Caltrans Amendments, Article 5.7.3.6. In computing the long-time deflection it shall be no less than the immediate deflection multiplied by a factor of 2.

12.11.3 Waterproofing of Underground Station Structures

Roofs, walls, and floors slabs of underground station including auxiliary spaces except as otherwise noted, shall be waterproofed. To ensure adequate inspection and long term performance, no blind side waterproofing shall be used.

Provisions shall be made to collect and drain water potentially seeping through the roof, walls, or floor. The leakage through structural elements shall be limited to a maximum of 0.001 gallon per square feet of structure per day, and no dripping or visible leakage from a single location shall be permitted.

The manufacturer and installer of the waterproofing system shall submit a list of a minimum of five successful projects of similar design and complexity completed within the past five years.

The designer shall design for any openings or other penetrations through the waterproofing layer and for appropriate protection measures for the waterproofing membrane including the chamfering of corners of the structure, external protection, etc. Components of the waterproofing system shall comply with applicable Volatile Organic Compound (VOC) regulations.

12.11.3.1 Underground Station Structures

A. Roofs

Station roofs shall be completely waterproofed. Waterproofing and the boundary condition details at reglets and flashings shall be provided.

B. Walls

Exterior station walls shall be completely waterproofed. Mezzanine walls enclosing public areas and entrance walls shall be furred out, and provisions shall be made for collecting and draining seepage through these walls. The depth of the furring shall be governed by the space required for the placing of fare collection and other equipment, and architectural requirements, such as the minimum thickness of the wall finish. The fastening of the finish to the wall shall be such that water can drain off the walls freely and that it will not corrode the fasteners.

C. Floor Slabs

For station floor slabs, no special waterproofing provisions shall be made where the water can drain freely into the floor drainage system, and where such a leakage and drainage is not objectionable from a corrosion, operational, or visual standpoint.

Drainage shall be provided at public areas of the station floor slab.

D. Base Slabs

Waterproofing shall be applied under station base slab.

E. Appendages

Differential vertical movements of the station body and its appendages, such as wings or entrances at shafts, due to ground re-expansion as a result of returning of ground water, may cause cracks at joints and other locations. Special attention shall be given to design detailing to mitigate this problem. Where such movements cannot be avoided, properly designed waterproof joints between such appendages and the station body shall be provided.

12.11.3.2 Cut-and-Cover Underground Trackway Structures

A. Cut-and-Cover Box

Exterior membrane waterproofing shall be applied to the outside of the cut-and-cover box as indicated on the Standard and Directive Drawings. Any seepage through the walls or the floor shall be carried away by the track drainage.

B. Transition Structure

For underground structure daylight transition structures, where U-sections or trenches with exposed sidewalls are used, special attention shall be given to controlling shrinkage cracks in sidewalls between construction joints.

C. <u>Rooms</u>

The following rooms or spaces shall be completely waterproofed, including all wall and roof surfaces in contact with the earth. Floor drains shall be provided. Refer to the *Mechanical* chapter.

- Electrical Rooms (includes spaces that house train control facilities, substation facilities, switchgear, ventilation fans, pumps, and other electrical equipment)
- Train Control and Auxiliary Equipment Rooms
- Substation , Switchgear, Fan Rooms, and Similar Equipment Rooms

D. <u>Pump Rooms</u>

Floor drains shall be provided to prevent the accumulation of seepage as required in the *Mechanical* chapter.

E. <u>Cross-passages and Emergency Exits</u>

Cross-passages shall be provided between tracks at every 2500 feet.

12.11.3.3 Waterstops and Sealants

Waterstops and sealants shall be used in construction joints in exterior walls, floors, and roofs.

12.11.3.4 Waterproofing Materials

Bentonite waterproofing shall not be used.

12.11.3.5 Water tightness

The cut-and-cover structure shall be designed and constructed so that it achieves a functional waterproofed underground structure for the duration of its design life. The design, construction, and maintenance of the cut-and-cover structure shall meet the water-tightness criteria stipulated below until substantial completion and acceptance by the Authority:

- Local infiltrations limit 0.002 gallons per square foot of structure per day and no dripping or visible leakage from a single location shall be permitted.
- No drips shall be permitted overhead or where they have the potential to cause damage to equipment, malfunctioning of any electrical power, signaling, lighting, control, communication equipment, or compromise electrical clearances.
- A drainage system shall be provided to accommodate water infiltration as specified herein in accordance with tunnel and portal drainage.
- No water ingress shall cause entry of soil particles into the tunnel.
- No material used in preventing or stemming water ingress shall compromise the fire safety of the works or the durability of the structures in which they are used.

- Embedded electrical boards, electrical conduits, and other similar elements shall be completely waterproofed and watertight.
- The interface between cut-and-cover structure section with bored tunnel and other structures, (i.e., building structures, emergency egress structures, etc.) shall be designed and constructed such that the joint between the two structures is fully watertight.

12.11.4 Water Holding and Conveyance Structures

Water conveyance or water holding structures that cross the HST alignment shall be designed to meet ACI 350 Code Requirements for Environmental Structures and Commentary with all Errata. The jurisdiction owning or operating the facility may have additional requirements that shall be followed.

12.11.5 Shoring Systems

For specific requirements of soil loadings see the *Geotechnical* chapter. The design of the support system shall consider several factors, including, but not limited to the following:

- Soil and groundwater conditions
- Width and depth of excavation
- Configuration of the structure to be constructed within the cut
- Size, foundation type and proximity of adjacent structures
- Utilities crossing the excavation, or adjacent to the excavation
- Requirements for traffic decking across the excavation
- Traffic and construction equipment surcharge adjacent to the excavation
- Settlements of adjacent structures
- Noise restrictions

12.11.6 Structural Fire Resistance

Underground structures can be exposed to extreme events such as fires resulting from incidents inside the structure. Underground structure design shall consider the effects of a fire on the concrete supporting elements. The concrete elements should be able to withstand the heat of the specified fire intensity given in the *Tunnels* chapter and period of time without loss of structural integrity. Protection from fire shall be determined by concrete cover on the reinforcing, additional finish, and special treatment of the concrete mixes.

12.12 Support and Underpinning of Structures

This section includes design requirements for the support and underpinning of existing structures to remain over or adjacent to new HST facilities.

The designer, in coordination with the Authority, shall investigate existing structures, which are to remain over, or adjacent to, the construction sites of new HST facilities. The designer shall prepare the necessary designs for the protection or permanent support and underpinning of such existing structures.

The types of buildings and structures, which require support and underpinning, include the following:

- Buildings and structures that extend over the HST structures to such an extent that they must be temporarily supported during construction and permanently underpinned.
- Buildings and structures immediately adjacent to the HST structures that will require temporary support during construction.
- Buildings and structures that are affected by groundwater lowering. In certain areas, uncontrolled lowering of the groundwater for HST construction can cause settlements of buildings either adjacent to or at some distance from HST excavations.

The design shall conform to the applicable requirements of the AASHTO LRFD with Caltrans Amendments (where highway bridges are involved), AREMA (where railway bridges are involved), CBC (where buildings are involved), ACI, AISC and AWS except where such requirements conflict with the criteria.

12.12.1 Depth of Support Structures

Underpinning walls or piers which support buildings or other structures and which also form a portion of the excavation support system shall extend to a minimum depth of two feet below the bottom elevation of the excavation.

12.12.2 Methods

Methods used to protect or underpin buildings or other structures shall take the site-specific soil conditions into consideration.

12.12.2.1 Protection Wall Method of Structure Protection

Under some soil conditions, the supporting system for the excavation is sufficient to protect light structures. Under heavier loading conditions, a reinforced concrete cutoff wall, constructed in slurry-filled trenches or bored pile sections braced with preloaded struts, may be considered as an alternative to underpinning or as a means to avoid settlement due to dewatering.

12.12.2.2 Stabilization of Soil

Soil stabilization techniques such as compaction grouting may be considered as alternatives in lieu of underpinning. Refer to *Geotechnical* chapter for soil stabilization.

12.12.2.3 Temporary Bracing Systems

A tight bracing system is important for the effectiveness of underpinning and for protection wall support. In addition to the general requirements for support of excavations, which are provided in the specifications, the designer shall indicate special requirements for the installation and removal of the temporary bracing systems that relate to the designs of underpinning and protection walls, such as the levels of bracing tiers, the maximum distances of excavation below an installed brace, and the amount of preloading. The designer shall require that detailed design of the temporary bracing system be the responsibility of the contractor. Refer to the *Geotechnical* chapter for earth pressures.

12.12.2.4 Pier, Pile, or Caisson Method of Underpinning

If soil conditions, structure size and proximity to an excavation dictate piers, piles or caissons for underpinning of an existing structure, such piers, piles, or caissons shall extend below a sloping plane which is defined as follows: The plane passes through a horizontal line which is located two feet below the bottom of the excavation, and which is also located within the vertical plane containing the face of that excavation closest to the structure foundation to be underpinned; the plane shall slope upwards and away from the excavation at an inclination which shall be established by the designer, on a case-by-case basis. The supports shall be founded on stable soil mass and extended beyond the slope of the soil wedge failure plane. Refer to the *Geotechnical* chapter for soils information.

12.12.2.5 Temporary Shoring and Underpinning

For temporary shoring and underpinning of the existing operating structures, seismic loads shall be considered in the shoring and underpinning design. The soil and inertia lateral seismic design loads shall be determined by geotechnical engineers using requirements. For seismic requirements of temporary shoring and underpinning, see the *Seismic* chapter. Shoring shall be required to maintain at-rest soil condition and monitored for movement.

12.13 Areas of Potential Explosion

Areas of new buildings adjacent to facilities where the public has access or that cannot be guaranteed as a secure area, such as parking garages and commercial storage and warehousing, shall be treated as areas of potential explosion.

NFPA 130, Standard for Fixed Guideway Transit and Passenger Rail Systems, life safety separation criteria shall be applied that assumes such spaces contain Class-I flammable or Class-II or Class-III combustible liquids. For structural and other considerations, separation and isolation for blast shall be treated the same as for seismic, and the more restrictive requirement shall be applied.

12.14 Structure Interface Issues

The design of aerial structures shall accommodate the requirements for safe operation of trains. Design of structures shall provide for interface with all other design elements.

12.14.1 Cable Trough

A cable trough shall be provided on both sides of the trackway. The cable trough shall be continuous through the entire system. The top of the cable trough shall be used as a safety walkway as a non-skid surface is provided.

12.14.2 Grounding and Bonding

Refer to the *Grounding and Bonding Requirements* chapter for grounding and bonding design of structures including, but not limited to the following:

- Aerial structures and bridges
- Trenches and retaining walls
- Cut-and-cover structures
- Buildings and support facilities
- New and existing third party structures
- Miscellaneous structures (e.g., cable trough, sound wall, etc.)

12.14.3 Drainage

Water shall be drained from the trackway and conveyed to the drainage system. See the *Drainage* chapter for drainage requirements.

12.14.4 Conduit Risers

Provisions shall be made for the installation of conduit up the sides of specific columns of aerial structures, walls of earth retaining structures, and walls of cut-and-cover structures. Further, details shall be provided for the installation such that no damage is inflicted on the columns, parapets, or walls.

12.14.5 Embedded Conduits

Sleeves shall be embedded in the cable trough and parapet to provide routing for future electric cable installation.

12.14.6 Trackside Equipment

Provisions shall be made to support trackside equipment. This equipment shall be located in line with the OCS poles.

12.14.7 Access Stairs

Access stairs shall be provided to the trackway from the ground surface in trenches, retained cuts, and cut-and-cover structures. The stairs and entrances shall be secured with fences and gates described in the *Civil* chapter. Minimum width of access stairs shall be 5 feet.

12.14.8 Overhead Concrete Anchors

These criteria apply to anchors for overhead applications and subjected to sustained tensile loads where failure of the anchor could result in risk to life or limb.

Anchors shall be embedded in confined concrete. Length of embedment in unconfined concrete shall not be considered effective embedment length.

Use of adhesive anchors in overhead applications or in sustained tension is prohibited.

12.14.9 Utilities

For utility requirements within HST structures, refer to the *Utilities* chapter.