California High-Speed Train Project

TECHNICAL MEMORANDUM

Structure Design Loads
TM 2.3.2

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Note: Signatures apply for the latest technical memorandum revision as noted above.

Prepared by PB

for the California High-Speed Rail Authority

System Level Technical and Integration Reviews

The purpose of the review is to ensure:

- Technical consistency and appropriateness
- Check for integration issues and conflicts

System level reviews are required for all technical memoranda. Technical Leads for each subsystem are responsible for completing the reviews in a timely manner and identifying appropriate senior staff to perform the review. Exemption to the system level technical and integration review by any subsystem must be approved by the Engineering Manager.

System Level Technical Reviews by Subsystem:

Systems:	NOT REQUIRED Print Name:	DD Month YY Date
Infrastructure:	*Signed document on file* Fletcher Waggoner	3 June 10 Date
Operations:	NOT REQUIRED Print Name:	DD Month YY Date
Maintenance:	NOT REQUIRED Print Name:	DD Month YY Date
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Note: Signatures apply for the technical memorandum revision corresponding to revision number in header and as noted on cover.

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ABSTRACT

This technical memorandum defines the permanent and transient load effects used in the Load and Resistance Factor Design (LRFD) of structures directly supporting dedicated high-speed trains for the California High-Speed Train Project (CHSTP). It is intended for the design of CHSTP bridges, aerial structures, and grade separations.

This technical memorandum shall not be used for design of non-aerial structures which carry high-speed trains. These include, but are not limited to underground structures, cut-and-cover structures, trenches, retaining structures, or at-grade sections. Although this memorandum currently provides design guidance for loads which could be deemed applicable to earth retaining structures and cut-and-cover structures, it is not currently intended to be comprehensive with regard to these structure types.

Facility loads, such as those for buildings and stations not supporting high-speed trains, are covered in TM 2.5.1: Structural Design of Surface Facilities and Buildings.

For structures carrying highway loads, Caltrans Bridge Design Specifications (CBDS) shall generally apply, with supplementary provisions herein which account for loads or performance criteria specific to high-speed train operations.

Specific criteria and evaluation measures have not been developed for existing structures which could impact operability of high-speed trains in event of failure. For example, procedures have not been developed for evaluation or assessment of an existing highway bridge which spans over a high-speed trackway. Future criteria will address this.

Requirements specific to train-structure compatibility and rail-structure interaction are addressed in TM 2.10.10: High-Speed Train and Track Structure Compatibility.

Technical memoranda are pending for high-speed train tunnel design and load effects.

At this time, the CHSTP high-speed rolling stock is unknown. In order to advance preliminary design, the Cooper E-50 based upon the AREMA Manual for Railway Engineering Specification has been selected as the design train.

This document is to be used in conjunction with the following technical memoranda:

- TM 2.10.4, Interim Seismic Design
- TM 2.9.10, Geotechnical Analysis and Design Criteria Guidelines
- TM 2.10.10, High-Speed Train and Track Structure Compatibility
- TM 3.2.2, OCS Structural Requirements
- TM 2.5.1, Structural Design of Surface Facilities and Buildings
- TM 2.1.5, Track Design

1.0 INTRODUCTION

1.1 PURPOSE OF TECHNICAL MEMORANDUM

This technical memorandum establishes the permanent and transient load effects to be considered for structures directly supporting dedicated high-speed trains. It is intended for the LRFD design of bridges, aerial structures, and grade separations.

1.2 STATEMENT OF TECHNICAL ISSUE

The definition of the permanent and transient load effects is necessary for LRFD methodology of CHSTP structures supporting high-speed trains.

1.3 GENERAL INFORMATION

1.3.1 Definition of Terms

Include technical terms, acronyms, foreign phrases/terms, etc. and or terminology that may have specific connotations with regard to the California High-Speed Train System.

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AREMA	American Railway Engineering and Maintenance-of-Way Association
Authority	California High-Speed Rail Authority
AWS	Structural Welding Standards
CBDA	Caltrans Bridge Design Aids Manual
CBDD	Caltrans Bridge Design Details Manual
CBDM	Caltrans Bridge Design Manuals
CBDP	Caltrans Bridge Design Practice Manual
CBDS	Caltrans Bridge Design Specifications: AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments
CBC	California Building Code 2007
CDC	CHST Project Design Criteria
CF	Centrifugal force
CHST	California High-Speed Train
CHSTP	California High-Speed Train Project
CL	Collision loads
CMTD	Caltrans Bridge Memo to Designers Manual
CSDC	Caltrans Seismic Design Criteria
CR	Creep effects
DBE	Design Basis Earthquake
DC	Dead load of structural components and permanent attachments
DD	Downdrag force
DR	Derailment loads
DW	Dead load of non-structural and non-permanent attachments
ED	Dynamic earth pressures
EH	Lateral static earth pressure
EL	Locked-in construction forces
ES	Surcharge loads
EV	Vertical earth pressure
FR	Frictional force
I	Vertical impact effect
LDBE	Lower-level Design Basis Earthquake
LF	Traction or braking forces
LLH	Highway Live Loads
LLS	Live Load Surcharge

LLP	Floor, roof, and pedestrian live loads
LLRR	Maintenance and construction train live load
LLV	High-speed train live loads
LRFD	Load and Resistance Factor Design
MCE	Maximum Considered Earthquake
NE	Nosing and hunting effects
PCF	Pounds per cubic foot
PS	Secondary Forces from Prestressing
PSF	Pounds per square foot
SE	Earth settlement effects
SH	Shrinkage effects
SS	Slipstream effects
TAP	Technical Advisory Panel
TG	Gradient temperature effects
THSR	Taiwan High Speed Rail
TU	Uniform temperature effects
WA	Water loads
WAD	Hydrodynamic force
WL	Wind load on live load
WS	Wind load on structure

1.3.2 Units

The California High-Speed Train Project (CHSTP) is based on U.S. Customary Units consistent with guidelines prepared by the California Department of Transportation (Caltrans) and defined by the National Institute of Standards and Technology (NIST). U.S. Customary Units are officially used in the U.S. and are also known in the U.S. as “English” or “Imperial” units. In order to avoid any confusion, all formal references to units of measure should be made in terms of U.S. Customary Units.

2.0 DEFINITION OF TECHNICAL TOPIC

2.1 GENERAL

This technical memorandum defines the permanent and transient load effects used for Load and Resistance Factor Design (LRFD) of CHSTP bridges, aerial structures, and grade separations.

This technical memorandum shall not be currently used for design of non-aerial structures which carry high-speed trains. These include, but are not limited to underground structures, cut-and-cover structures, earth retaining structures, or at grade sections.

2.2 POLICY CONSIDERATIONS

The following policy considerations affect this technical memorandum.

2.2.1 General Classifications

CHSTP structures are generally classified as:

- Bridges – high-speed train trackway structures spanning rivers, lakes, canals, highways, railroads, and canyons
- Aerial Structures – elevated high-speed train trackway structures
- Grade Separations – structures separating high-speed train trackways from highway or pedestrian usage
- Earth Retaining Structures – including U-walls and retaining walls
- Cut-and-Cover Underground Structures – including cut-and-cover underground stations or track structures
- Bored Tunnel Linings
- Mined Tunnels
- Buildings and All Other Above Ground Structures – 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

CHSTP facilities, based on their importance to high-speed train service, are classified as Primary or Secondary Structures.

Primary Structures: Primary Structures are those that directly support track and running trains, including bridges, aerial structures, stations, tunnels and underground structures, earth retaining structures, and embankments. Primary Structures include facilities essential to train service including train control, operation, communication, traction power, power distribution network, equipment, and maintenance facilities. Primary Structures also include systems essential to train service including tracks, rail fasteners, slab track, and ballast.

Secondary Structures: Secondary Structures are those that are not necessary for immediate resumption of train service including administrative buildings, shop buildings, storage facilities, parking structures and training facilities.

This document is related to design of Primary Structures.

2.2.2 Structural Design Parameters

1. All structures shall be designed for the appropriate loadings and shall comply with the structure gauge adopted for the high-speed train system.
2. Structure design load assumes dedicated high-speed rail operations. Freight rail vehicles will not operate on high-speed rail lines.

3. Structural design guidance shall apply to all structures adjacent to, above, or below the high-speed tracks. This includes structures carrying high-speed trains and newly constructed highway or ancillary structures, should they directly affect high-speed rail operations. For example, a newly constructed highway bridge above or adjacent to a high-speed rail bridge shall meet CHSTP seismic performance criteria for the MCE event, since its potential failure would directly affect high-speed rail operations.
4. The design life of fixed facilities shall be 100 years. Elements that are normally replaced for maintenance, such as expansion joints, may be designed to a shorter term.
5. The maximum design speed for the main tracks is 250 miles per hour; segments of the alignment will be designed to lesser speeds.
6. The bridges and aerial superstructures shall be designed as rigid and stiff in order to meet serviceability and comfort requirements for high-speed train operation.
7. Design and construction of high-speed train facilities shall comply with the approved and permitted environmental documents.

2.2.3 Seismic Design Parameters

For seismic design guidance and performance, refer to TM 2.10.4: Interim Seismic Design.

Based upon the structure's importance and technical classification, it may need to comply with multiple seismic performance criteria.

TM 2.10.6: Surface Fault Rupture Analysis and Design requires that tracks cross major capable fault zones at-grade, with near perpendicular alignment to the fault trace.

Project specific seismic criteria is pending oversight by a Technical Advisory Panel (TAP).

2.3 LAWS AND CODES

Initial high-speed train (HST) design criteria will be issued in technical memoranda that provide guidance and procedures to advance the preliminary engineering. When completed, a Design Manual will present design standards and criteria specifically for the design, construction and operation of the CHSTP's high-speed railway.

Criteria for design elements not specific to HST operations will be governed by existing applicable standards, laws and codes. Applicable local building, planning and zoning codes and laws are to be reviewed for the stations, particularly those located within multiple municipal jurisdictions, state rights-of-way, and/or unincorporated jurisdictions.

In the case of differing values, the standard followed shall be that which results in the satisfaction of all applicable requirements. In the case of conflicts, documentation for the conflicting standard is to be prepared and approval is to be secured as required by the affected agency for which an exception is required, whether it be an exception to the CHSTP standards or another agency standards.

3.0 ASSESSMENT / ANALYSIS

3.1 GENERAL

This technical memorandum defines the permanent and transient load effects used in the Load and Resistance Factor Design (LRFD) of structures directly supporting dedicated high-speed trains for the California High-Speed Train Project (CHSTP).

Structures shall be designed to resist temporary construction stages and loadings. Where applicable, design shall consider soil-structure interaction, as well as temporary construction loads during staged construction.

For buildings and stations not supporting high-speed trains, reference TM 2.5.1: Structural Design of Surface Facilities and Buildings.

For structures carrying highway loads, Caltrans Bridge Design Specifications (CBDS) shall generally apply, with supplementary provisions herein which account for loads or performance criteria specific to high-speed train operations.

Additional requirements specific to train-structure compatibility and rail-structure interaction are addressed in TM 2.10.10: High-Speed Trains and Track Structure Compatibility.

Technical memoranda are pending for high-speed train tunnel design and load effects.

3.2 DESIGN CODES AND SPECIFICATIONS

Design shall meet all applicable portions of the general laws and regulations of the State of California and of respective local authorities. The design of the CHST structures use Load and Resistance Factor Design (LRFD) methodology.

Unless otherwise specified, the CHSTP facilities shall be designed in accordance with applicable portions of the following standards and codes:

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. ACI: American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318-05
3. AISC: American Institute of Steel Construction, Steel Construction Manual, 13th Edition
4. AWS D1.1/D1.1M:2008 Structural Welding Code-Steel
5. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code
6. AWS D1.8/D1.8M:2009 Structural Welding Code-Seismic Supplement
7. CBC: The 2007 California Building Code
8. California Department of Transportation (Caltrans) Bridge Design Manuals, latest edition
   - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments.
   - Bridge Memo to Designers Manual (CMTD)
   - Bridge Design Practices Manual (CBPD)
   - Bridge Design Aids Manual (CBDA)
   - Bridge Design Details Manual (CBDD)
   - Bridge Memo to Designers Manual (CMTD)
   - Standard Specifications
   - Standard Plans
   - Seismic Design Criteria ver. 1.5 (CSDC)

In addition, the design of structures to be built as part of the CHSTP but owned by other agencies or private owners, shall meet the requirements of the agencies which normally have jurisdiction over such. In the case of differing values, the standard followed shall be that which results in the satisfaction of all applicable requirements. In the case of conflicts, documentation for the conflicting standard is to be prepared and approval is to be secured as required by the affected agency for which an exception is required, whether it be an exception to the CHSTP standards or another agency's standards.

3.3 DESIGN REFERENCES

This technical memorandum also uses information drawn from the following references:

1. European Standard EN 1991-2:2003 Traffic Loads on Bridges
2. European Standard EN 1991-1-7:2006: Accidental Actions
3. European Standard EN 1990 annex A2: Application to Bridges
4. Taiwan High Speed Rail (THSR) Corporation: Volume 9, Sections 1, 3, and 9

3.4 PERMANENT LOADS

3.4.1 Dead Load (DC, DW)

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

In the absence of more precise information, the unit weights specified in Table 3-1 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, walkways, sound walls, overhead contact system, etc.

DW refers to the dead load of non-structural attachments which may be non-permanent including, wearing surfaces, utilities, cable trays, finishes, etc.

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

Table 3-1 – Unit Weights of Common Materials

Item		Unit Weight	Reference
Aluminum alloys		175 pcf	CBDS
Ballast, Rolled Gravel, Macadam		140 pcf	CBDS
Bituminous Wearing Surfaces		140 pcf	CBDS
Cast Iron		450 pcf	CBDS
Cinder Filling		60 pcf	CBDS
Compacted Sand, Silt, or Clay		120 pcf	CBDS
Unreinforced Concrete	Lightweight	110 pcf	CBDS
	Sand-Lightweight	120 pcf	CBDS
	Normal Weight with f'c ≤ 5 ksi	145 pcf	CBDS
	Normal Weight with 5 < f'c ≤ 15 ksi	(140 + f'c) pcf	CBDS
Electrification (overhead system and fastenings)		100 pounds per foot of track	CHSTP
Loose Sand, Silt, Gravel or Soft Clay		100 pcf	CBDS
OCS poles/precast duct bank including walkway surface and walkway barrier		300 psf	THSRC
OCS wire break		13.3 kip in direction of intact wire each track	TM 3.2.2 OCS Structural Requirements
Rails and fasteners (no ties)		200 pounds per foot of track	AREMA
Slab track & slab track base		10 inches of concrete between OCS poles or curbs. This will be adjusted if a proprietary slab track is selected.	Slab track manufacturers
Soils		See Geotechnical recommendations	
Steel		490 pcf	CBDS
Stone Masonry		170 pcf	CBDS
Systems cable tray		180 pounds per foot of track	CHSTP
Water	Fresh	62.4 pcf	CBDS
	Salt	64.0 pcf	CBDS
Wood	Hard	60 pcf	CBDS
	Soft	50 pcf	CBDS

3.4.2 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 Design Report.

3.4.3 Earth Pressures (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 Design Report.

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 Design Report shall be used.

Lateral Static Earth Pressure (EH): For structures retaining draining cohesionless (granular) soil, lateral earth pressure shall be determined in accordance with the following paragraphs of these criteria. For structures retaining other soil types, the definitions shall be provided for those soil types and shall be included in the Geotechnical Design Report.

Yielding Walls: Yielding walls (i.e.: typical cantilever retaining walls) are defined as walls which are unrestrained and free to move at the top for a distance of at least 0.004H, where H is defined as the height of the wall from the base of the heel to the finished grade directly above the heel.

For yielding walls, the lateral static earth pressure shall be determined using the active lateral pressures expressed as equivalent fluid pressures. Recommended values given in the Geotechnical Design Report shall be used.

Rigid Walls: Rigid walls are defined as walls which are restrained at the top so that the amount of deflection required to develop active pressure conditions is not possible. All permanent excavation support walls as well as tunnel portal transitions and U-sections shall be considered rigid walls.

For rigid walls, the static lateral soil pressure shall be determined using the at-rest lateral pressures expressed as equivalent fluid pressures. Recommended values given in the Geotechnical Design Report shall be used.

3.4.4 Earth Surcharge (ES)

Surcharge loads (ES) are vertical or lateral loads resulting from loads applied above or below the adjacent ground surface. Procedures for determining surcharge load shall be recommended in the Geotechnical Design Report.

Underground structures and walls shall be designed for loading from existing adjacent buildings or structures. Consideration shall be given to the maximum and minimum loads that can be transferred to the design structure, and design loads shall be assumed to be the same as those for which the adjacent structure was designed. In the absence of this information, loads based on provisions in the CBC concurrent with the heaviest occupancy for which the building is suitable, shall be used.

Surcharge loads from adjacent buildings and structures commonly result from combinations of dead load, live load, etc. Due to this, surcharge loads applied for the design structure need not be factored twice, and shall be taken as the greater of the unfactored surcharge loading multiplied by the specified factor, ES, or the factored surcharge loading with ES equal to 1.0.

Structures that may support future developments above CHSTP facilities shall be designed to reflect the future load. Surcharge loads modeling future developments shall not be applied concurrently with traffic or future traffic surcharges in the same locations. If a future development extends to the surface land adjacent to an underground structure, then a surcharge shall be applied for design. The surcharge shall model the loading due to the maximum development currently allowed by zoning.

Construction activities may result in permanent or temporary loads from equipment, trucks, drayage, stockpiling of materials, or excavated earth. A surcharge shall be applied to all underground structures, unless: 1) positive and recognizable means are provided at the ground surface to ensure that the above types of loadings cannot occur; and, 2) the Authority specifically permits in writing, the application of a surcharge of lesser magnitude.

The vertical surcharge shall be considered as a static uniform load as follows:

600 psf, for $h < 5$ ft

600-40 (h-5) psf, for $5 \text{ ft} \le h \le 20$ ft

0 psf, for $h > 20$ ft

h = vertical distance from the top of structure to ground surface.

Surcharge loads representing construction activities need not be applied concurrently with traffic or future traffic surcharges in the same location.

See Article **Error! Reference source not found.** Live Load Surcharge (LLS).

3.4.5 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 Design Report shall be used.

Tolerable settlement on foundations shall be developed consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on structural and operational performance.

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

Specific settlement criteria will be developed at a later date.

3.4.6 Creep Effects (CR)

For the effects due to creep of concrete (CR), the requirements in CBDS Section 5.4.2.3 shall be used.

3.4.7 Shrinkage Effects (SH)

For the effects due to shrinkage of concrete (SH), the requirements in CBDS Section 5.4.2.3 shall be used.

3.4.8 Secondary Forces from Prestressing (PS)

Secondary forces from prestressing (PS) shall be considered. Such secondary forces arise during prestress of statically indeterminate structures, which produce internal forces and support reactions. Secondary forces are generally obtained by subtracting the primary prestress forces from the total prestressing.

3.4.9 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 and bracing at excavation supports.

3.4.10 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 CBDS Section 3.7. Recommended values given in the Geotechnical Design Report shall be used.

The effects of ground water hydrostatic force, including buoyancy (WA) shall be considered in the design of underground structures and the substructure of aerial structures and buildings, including foundations and piling.

Adequate resistance to flotation shall be provided at all sections for full uplift pressure on the foundation, based upon the maximum probable height of the water table defined in the Geotechnical Design Report. For the completed structure, such 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 all joints). Friction acting on rigid excavation support systems such as diaphragm walls in permanent structures may also be used to resist uplift.

Hydrostatic pressure shall normally be applied to all surfaces in contact with groundwater with a magnitude based on the depth of water and the applicable water density.

Changes in foundation condition due to scour shall be investigated per CBDS Article 3.7.5.

3.5 TRANSIENT LOADS

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

Live loads are due to high-speed trains, Cooper E-50 maintenance trains, highway loads, construction equipment, and pedestrians.

3.5.1.1 Floor, Roof, and Pedestrian Live Loads (LLP)

For the force effects due to floor, roof, and pedestrian live loads (LLP), reference TM 2.5.1: Structural Design of Surface Facilities and Buildings. Technical Memorandum 2.5.1 also includes provisions for aerial trackway supporting service walkways.

3.5.1.2 High-Speed Train Live Loads (LLV)

The project specific rolling stock has not yet been determined. Once the project specific rolling stock is determined, the live load criteria will be expanded to consider these design trains.

Where a structure supports multiple tracks, the loading shall be applied for those number of tracks either simultaneously or individually, whichever governs design.

3.5.1.3 Maintenance and Construction Train Live Loads (LLRR)

Structures shall be designed to support maintenance and construction trains, defined as the Cooper E-50 in the AREMA Specification, see Figure 3-1.

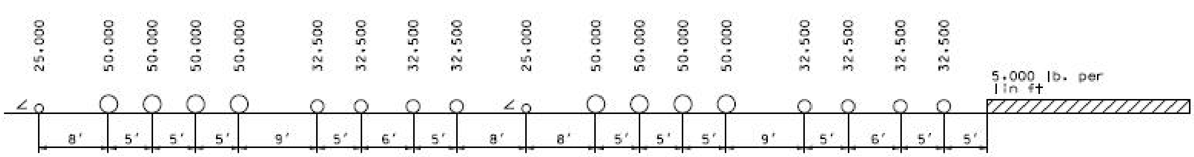


Figure 3-1: Cooper E-50 (LLRR)

For the case of multiple tracks on the bridge, LLRR shall be as follows (AREMA Chapter 15, Article 1.3.3(d)):

- 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.

3.5.1.4 Highway Live Loads (LLH)

Facilities designed to support highway loads (LLH) shall be designed for a minimum of HL-93 Truck Loading based on CBDS Section 3.6.1. For facilities that support highway permit loads, Caltrans guidelines shall be followed for the routing and sizes of the permit vehicles.

For underground structures beneath city streets or planned roadways including culverts, spread footings and pile caps, the distribution of live load shall be in accordance with the following:

- Fill height less than two feet: Live load shall be applied as concentrated wheel loads directly to the top of the slab in addition to the uniform load specified in HL-93 loading.
- Fill height greater than two feet: Concentrated wheel loads shall be distributed over the contact area, and increased by the depth of fill and added to the HL-93 uniform load.
- When distribution areas overlap, the total load shall be uniformly distributed over an area defined by the outside limits of the individual areas.
- The design lane load shall be distributed over that portion of a lane width as described in CBDS.

3.5.1.5 Live Load Surcharge (LLS)

An area surcharge (LLS) shall be applied at the ground surface both over and adjacent to underground and retaining structures, as applicable, to account for presence of live load. Live load surcharge may result 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, Chapter 8, Article 5.3.1 & 16.4.3. Lateral distribution of highway surcharge shall be in accordance with CBDS, Article 3.6.1.2.6. No impact factors apply to LLS for walls. Buried components shall be subject to impact as follows:

$$I(buried) = I * (1.0 - 0.125D) \geq 0\%$$

Where:

I = impact in accordance with Article **Error! Reference source not found.** herein

D = minimum depth of earth cover above structure (ft)

Recommended coefficients for lateral surcharge loading shall be as recommended in the Geotechnical Design Report.

3.5.1.6 Live Loads 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 all structures.

The fatigue assessment shall be performed for all structural elements which are subjected to fluctuations of stress. For structures supporting multiple tracks the loading shall be applied to a minimum of two (2) tracks in the most unfavorable positions. The fatigue damage shall be assessed over the required structural life of 100 years.

Fatigue assessment criteria is pending the next revision of this technical memorandum.

3.5.2 Vertical Impact Effect (I)

Moving trains and vehicles impart dynamic load 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.

Impact applies to the following:

- Superstructures, 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 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).

LLRR

Static impact factors (I) for LLRR shall be taken from AREMA and Eurocode Specifications.

Ballasted track:

Reinforced or prestressed concrete bridges: (AREMA Chapter 8, Section 2.2.3(d)):

$$I = \frac{225}{\sqrt{L}}$$ where 14 ft < L ≤ 127 ft

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

Steel bridges (AREMA Chapter 15, 1.3.5(c)):

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

$$I = 16 + \frac{600}{L-30}$$ where L ≥ 80 ft

For direct fixation on concrete slab track with spans less than 40 feet, European Standard EN 1991-2 shall be used as modified below. For longer spans, AREMA ballasted track impact factors shall be used as a lower bound.

For direct fixation on concrete slab track:

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

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

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

An additional ±20% imbalance of live load shall be applied to each rail as a vertical force to model the couple caused by 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.

LLV

Dynamic analysis is required for structures carrying high-speed trains (LLV) in order to determine impact effects.

For more dynamic analysis requirements, see TM 2.10.10: High-Speed Train and Track Structure Compatibility.

LLH

For determining impact factors (I) associated with highway loading (LLH), CBDS Section 3.6.2 shall be used.

3.5.3 Centrifugal Force (CF)

For structures carrying high-speed trains on a curved alignment, centrifugal force (CF) shall be considered. Multiple presence factors may apply to centrifugal forces.

LLRR and LLV

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

For LLRR, CF acts at 8 feet above top of rail.

For LLV, CF acts at 6 feet above top of rail.

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

V = train speed (mph)

R = horizontal radius of curvature (ft)

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 cant shall be considered when present. The cant 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.

LLH

For structures carrying highway loads on a curved alignment, centrifugal force (CF) shall be per CBDS Section 3.6.3.

3.5.4 Traction and Braking Forces (LF)

LLRR

For traction and braking forces (LF) from maintenance and construction trains (LLRR) use AREMA Section 2.2.3:

Traction force = $5/8(25\sqrt{L})$ kips, acting 3 feet above top of rail

Braking force = $5/8(45 + 1.2L)$ kips, acting 8 feet above top of rail

where L = length in feet of portion of bridge under consideration.

The LF loads for LLRR are to be distributed equally over the length of the train. Multiple presence factors shall apply.

LLV

For traction and braking forces (LF) from high-speed trains (LLV) use THSR Section 1, Appendix D, or European Standard 1991-2, Article 6.5.3:

Traction force = 2.26L (kips) or 25% of train load (if known), with a maximum value of 225kips , acting at top of rail

Braking force = 1.37L (kips) or 25% of train load (if known), with a maximum value of 1350kips, acting at top of rail where L = length in feet of portion of bridge under consideration.

These loads are to be distributed equally to all axles over the length of the train.

Note that traction and braking forces will be reviewed and possibly revised when project-specific rolling stock is selected.

LLH

For braking forces (LF) from highway loading (LLH), CBDS Section 3.6.4 shall be used.

3.5.5 Nosing and Hunting Effects (NE)

LLRR and LLH

Nosing and hunting effects do not apply to LLRR and LLH.

LLV

For structures carrying high-speed trains with slab track and direct fixation fasteners, nosing and hunting effects (NE) of the wheels contacting the rails shall be accounted by a 22 kip force applied to the top of the low rail, perpendicular to the track centerline at the most unfavorable position.

NE is not applicable for the design of structures with ballasted track.

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

3.5.6 Wind Loads (WS, WL)

LLRR and LLV

Wind Pressure on Structures (WS) and Wind Pressure on Trains (WL) shall be calculated in accordance with requirements in CBDS Section 3.8 with the following modifications:

(1) 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 all parapets of significant height contained on the bridge).

(2) Use of “Suburban” and “City” exposure categories to calculate design wind velocity is discouraged and shall be accepted only on a case-by-case basis.

(3) The base lateral load for Wind Pressure on Vehicles (WL) shall be revised to 0.300 klf perpendicular to the train acting 8 feet above the top of rail. CBDS 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.

(4) For structures which utilize soundwalls or windwalls 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 which is shielded by the wall, assuming the train to be 14 feet tall with a lower border at 1 foot from top of rail. However this reduction shall not exceed 50% of WL.

Local design of elements such as parapets or components on structures shall give consideration to wind loading and slipstream effects. Wind loading on these elements may be calculated per applicable building code, as detailed in TM 2.5.1: Structural Design of Surface Facilities and Buildings. 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).

LLH

Wind loads (WS, WL) on highway structures and vehicles shall be per CBDS Section 3.8.

3.5.7 Slipstream Effects (SS)

LLV

3.5.7.1 Aerodynamic Actions from Passing Trains

The passing of high-speed trains subjects any structure situated near the track to travelling waves of alternating pressure and suction. 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 travelling wave of alternating pressure and suction (see Figures 3-2 to 3-5). 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 head and rear ends of a train, when checking ultimate and serviceability limit states and fatigue. Equivalent loads are given in Sections 0 to 0.
- In Sections 0 to 0 the Maximum Design Speed V [mph] should 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 16.5 feet from the start and end of the structure measured parallel to the tracks, the equivalent loads in Sections 0 to 0 should be multiplied by a dynamic amplification factor of 2.0.

NOTE: For dynamically sensitive structures the above dynamic amplification factor may be insufficient and may need to be determined by a special study. The study should take into account dynamic characteristics of the structure including support and end conditions, the 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.

3.5.7.2 Simple Vertical Surfaces Parallel to the Track (e.g. noise barriers)

(1) Equivalent loads, $\pm q_{1k}$, are given in Figure 3-2

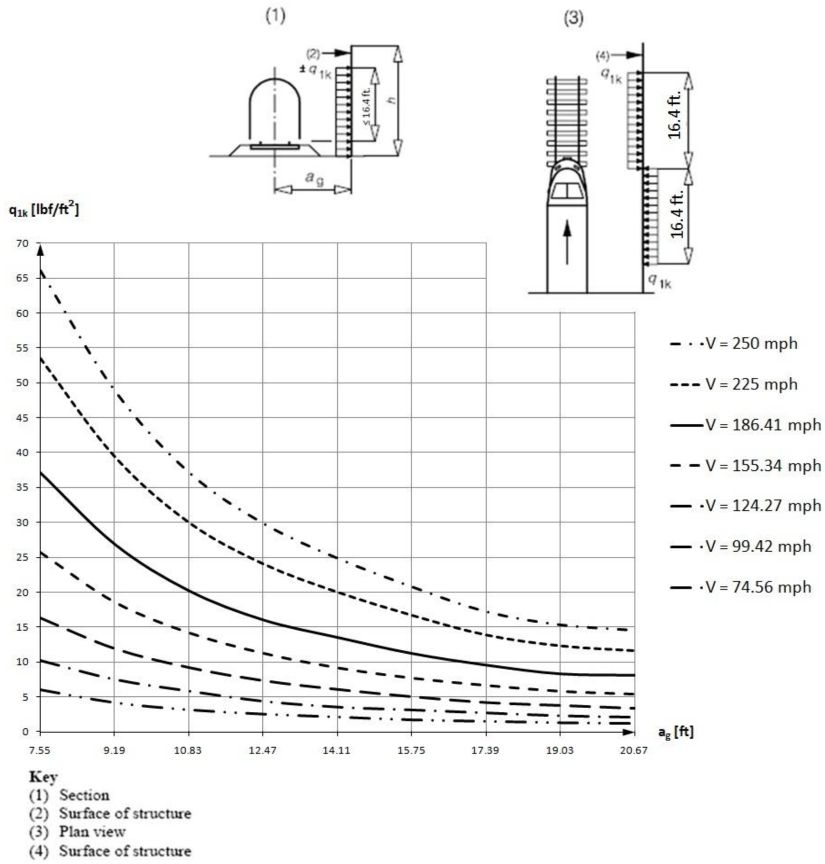


Figure 3-2: Equivalent loads q_{1k} for simple vertical surfaces parallel to the track

(2) 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)

(3) 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 noise protection wall, the actions q_{1k} should be increased by a factor k_2 = 1.3.

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

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

(2) The loaded width for the structural member under investigation extends up to 33 feet to either side from the centerline of the track.

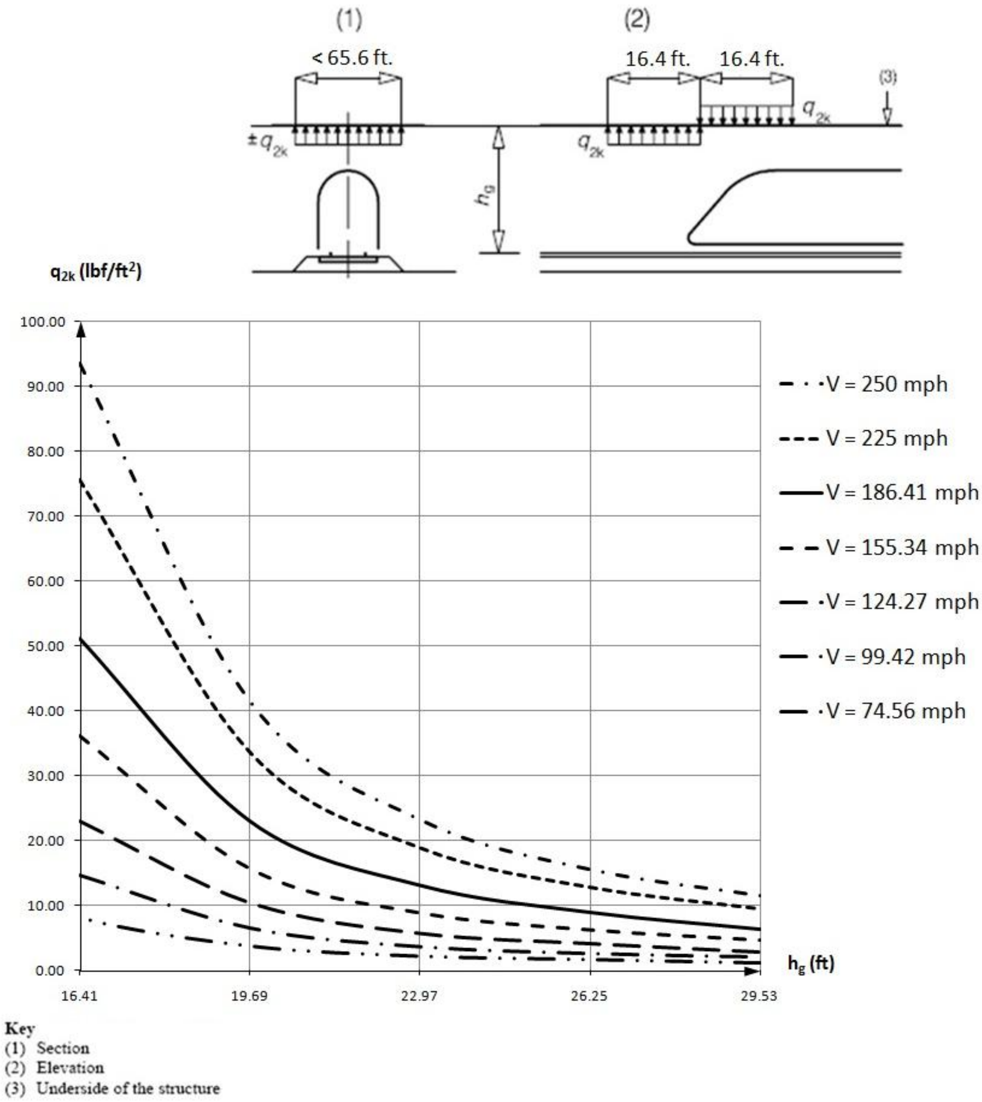


Figure 3-3: Equivalent loads q_{2k} for simple horizontal surfaces above the track

(3) For trains passing each other in opposite directions the actions should be added. The loading from trains on only two tracks needs to be considered.

(4) The actions q_{2k} may be reduced by the factor k_1 as defined in Section 0.

(5) The actions acting on the edge strips of a wide structure crossing the track may be multiplied by a factor of 0.75 over a width up to 5 feet.

3.5.7.4 Simple Horizontal Surfaces Adjacent to the Track (e.g. platform canopies with no vertical wall)

(1) Equivalent loads, $\pm q_{3k}$, are given in Figure 3-4 and apply irrespective of the aerodynamic shape of the train.

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

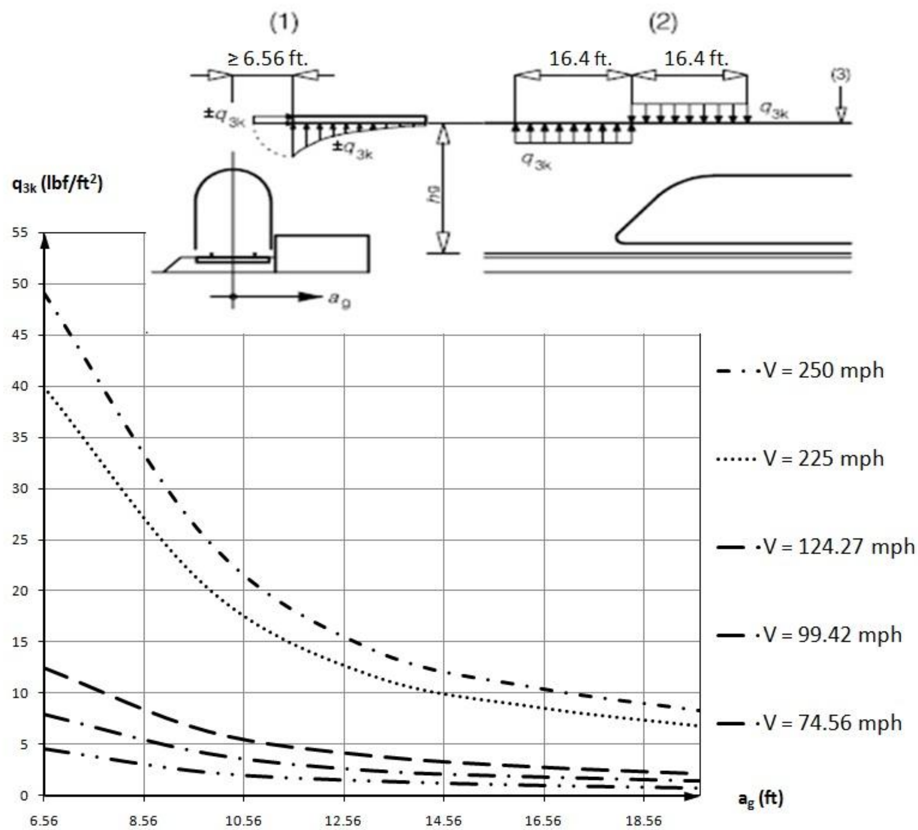
(3) If the distance h_g exceeds 12.5 feet the action q_{3k} may be reduced by a factor k_3:

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

$$k_3 = 0 \quad \text{for } h_g \geq 24.6 \text{ ft}$$

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

Figure 3-4: Equivalent loads q_{3k} for simple horizontal surfaces adjacent to the track

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

(1) Equivalent loads, $\pm q_{4k}$, as given in Figure 3-5 should be applied normal to the surfaces considered. The actions should be taken from the graphs in Figure 3-2 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 3-5

(2) If max a_g > 20 feet the value max a_g = 20 feet should be used

(3) The factors k_1 and k_2 defined in Section 0 should be used

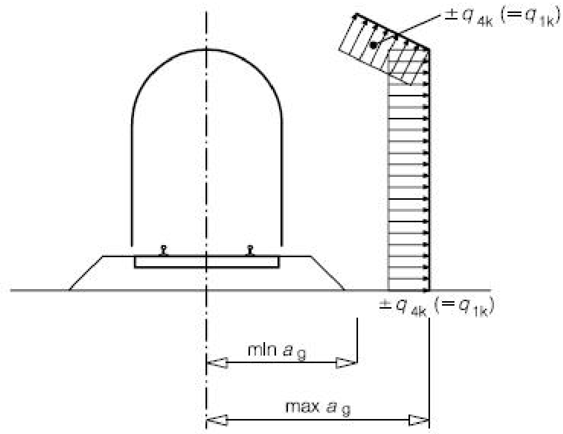


Figure 3-5: Definition of the distances min a_g and max a_g from center-line of the track

3.5.7.6 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)

(1) All actions should 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 0

  $k_4 = 2$

- to the horizontal surfaces:

  $\pm k_5 q_{2k}$

  where:

  q_{2k} is determined according to Section 0 for only one track,

  $k_5 = 2.5$ if one track is enclosed,

  $k_5 = 3.5$ if two tracks are enclosed

Additional analyses need to be developed and tested for final design.

3.5.8 Thermal Load (TU,TG)

For uniform (TU) and gradient (TG) temperature effects of the structure, the requirements in CBDS Section 3.12 shall be used.

For rail thermal ranges, including rail setting temperatures, reference TM 2.1.5: Track Design.

3.5.9 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.

3.5.10 Seismic Loads (MCE, DBE, LDBE)

Detailed, project specific seismic design criteria are presented in TM 2.10.4: Interim Seismic Design Criteria. TM 2.10.4 defines seismic design philosophies, seismic analysis/demand methodologies, and structural capacity evaluation procedures for the multiple levels of design earthquakes.

In general, an individual structure may need to comply with multiple performance levels, depending upon its Importance and Technical Classification.

Highway bridges spanning over high-speed train trackways, or highway structures which have the capacity to influence operability of high-speed trains in event of a failure, shall be designed for the Maximum Considered Earthquake (MCE) seismic event per provisions in TM 2.10.4: Interim Seismic Criteria, in addition to Caltrans Seismic Design Criteria (CSDC).

3.5.11 Hydrodynamic Force (WAD)

Hydrodynamic pressure effects acting on submerged portions of structures due to seismic motion shall be computed using the method of Goyal and Chopra [12] or by equivalent means.

For possible additional hydrodynamic force effects, see the Geotechnical Design Report.

3.5.12 Dynamic Earth Pressures (ED)

Dynamic earth pressure due to seismic motion on retaining structures shall be computed using the methods by Mononobe-Okabe analysis [13].

For possible additional dynamic earth pressure, see the Geotechnical Design Report.

3.5.13 Derailment Loads (DR)

3.5.13.1 LLRR and LLV

In the event of a derailment, the damage to the structures carrying high-speed trains shall be minimal. Overturning or collapse of the structure shall not be allowed, but local damage can be tolerated.

Two 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 an upstand 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 nonstructural elements such as walkways).

For Case I, collapse of a major part of the structure shall be avoided. Local damage, however, may be tolerated. The parts of the structure concerned shall be designed for the following design loads in the Extreme Loading Combination:

- Cooper E-50 loading (both point loads and uniformly distributed loading) 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 centerline of the track, or as limited by containment walls. If a short 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 short wall.

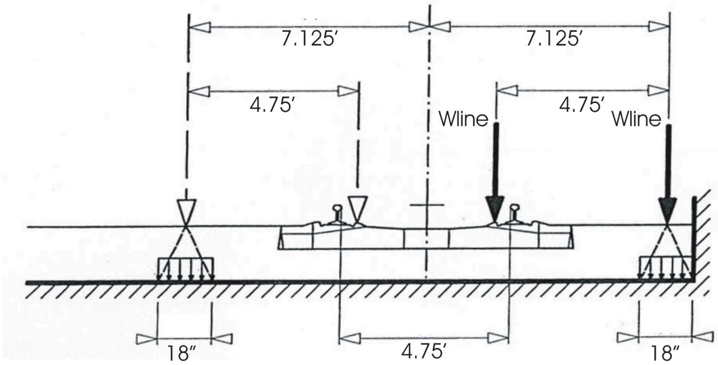


Figure 3-6: Derailment Case I

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

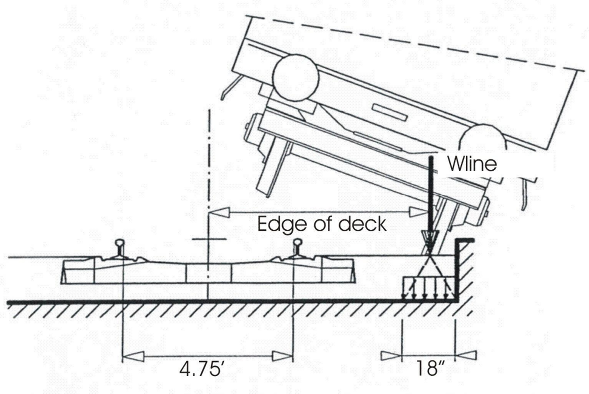


Figure 3-7: Derailment Case II

Design Situations 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 Figures 3-6 and 3-7.

For Design Situations I and II, other rail traffic actions should be neglected for the track subjected to derailment actions. When the structure under consideration carries more than one track, only one 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 apply in this case.

No dynamic factor needs to be applied to the derailment loads; however the loads shall be multiplied by the load factor within load combinations. A load factor of 1.0 shall apply to the horizontal force and a load factor of 1.4 shall apply to the vertical component.

For structural elements which are situated above the level of the rails, measures to mitigate the consequences of a derailment shall be in accordance with the specified requirements in addition to the above.

3.5.13.2 Track Side Containment

Derailment containment walls shall be provided for all mainline aerial structures at locations 7 feet or less from the track centerline. The height of the wall shall be minimum 0.67 feet above the level of the adjacent track's lower rail. A transverse horizontal concentrated load of 35 kips shall be applied at top of the wall.

3.5.14 Collision Loads (CL)

Collision loads in Sections **Error! Reference source not found.** to 3.5.14.2 apply to train impact loads (LLRR, LLV). Section 3.5.14.3 applies to highway collision loads (LLH). For collision loads on columns or divider walls along the trackway, reference TM 2.5.1: Structural Design of Surface Facilities and Buildings.

3.5.14.1 Collision Loads other than at Stations or Platforms

Members within 16.5 feet of the track center line shall be designed to resist train collision, 900 kips parallel to track, or 350 kips perpendicular to tracks.

3.5.14.2 Structures in Areas beyond Track Ends

(1) Overrunning of rail traffic beyond the end of a track or tracks (for example at a terminal station) should 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.

(2) The measures to manage the risk should 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.

(3) Supporting structural members to structures should generally not be located in the area immediately beyond the track ends.

(4) Where supporting structural members are required to be located near to track ends, an end impact wall should be provided in the area immediately beyond 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 engines. It is recommended that these forces are applied horizontally and at a level of 3 feet above track level.

3.5.14.3 Highway Vehicle Collision Loads (LLH)

Highway collision loading shall be as per CBDS Section 3.6.5.

3.6 MISCELLANEOUS LOADS

3.6.1 Overhead Contact System (OCS) Loads

Design of OCS poles and foundations shall be in accordance with TM 3.2.2: OCS Structural Requirements.

Loads shall include those arising from normal service, dead load, wind load, slipstream effects, seismic load, and unbalanced loads due to wire installation or wire breakage.

Deflection requirements shall be met as outlined in TM 3.2.2.

Catenary pole foundations shall be capacity-protected to prevent failure or unrecoverable damage to the bridge deck.

Aerial trackway structures shall be designed in such a manner that the OCS pole foundation locations can be determined at a later time.

3.6.2 Construction Loads and Temporary Structures

3.6.2.1 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 high-speed trains and/or pass over routes carrying high-speed trains. Subsequent articles herein apply to Type A structures.
- Type B: Temporary structures or structures under temporary conditions which do not carry high-speed trains and do not pass over routes carrying high-speed trains. These structures shall be designed in accordance with the specifications of the owning/operating agency (*e.g.* Caltrans: CBDS and CMTD). Structures such as haul

bridges used temporarily for the CHSRP shall be designed in accordance with CMTD 15-14.

3.6.2.2 Construction Load Combinations

Temporary structures or structures under temporary conditions shall be designed to adequately resist conditions at all stages of construction, including all applicable construction loads. Construction load combinations 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%.
- Service 1, as applicable.
- Seismic load combinations: For 30% design purposes, Load Combination Extreme 3, with revised ground motions having 10% probability of exceedance in 10 years. Performance criteria shall comply with the Safety Performance Level (SPL) for the Design Basis Earthquake (DBE) for an important, standard structure, see TM 2.10.4: Interim Seismic Design Criteria. If significant construction loads are anticipated or specialized equipment present, this mass shall be accounted for in the seismic design.

In the absence of better criteria, a construction live load of 10 psf shall be assumed on the bridge deck.

3.6.2.3 Segmental Construction and Specialized Equipment

Construction load combinations per CBDS Article 5.14.2 “Segmental Construction” shall be considered. The temporary seismic load event as detailed herein shall be added to the construction load combination at strength limit states; however a 1.0 load factor may 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.

3.6.3 Rail-Structure Interaction Forces

Effects of continuous welded rail (CWR) interaction with the structure through its attachment shall be considered. Design guidance for this interaction is provided in Technical Memorandum 2.10.10: High-Speed Train and Track Structure Compatibility.

Interaction effects of the CWR and structure result after connection of the rail to the structure. These occur in large part due to the following: temperature variations in the rail and structure, braking/traction, creep and shrinkage, and seismic events. Load combinations presented in this document assume this interaction implicit to each load within the combination, with associated load factors.

Structure thermal ranges shall be calculated per provisions in this document. For specific rail information, including thermal ranges, setting temperature, and rail section, reference TM 2.1.5: Track Design.

TM 2.10.10: High-Speed Train and Track Structure Compatibility also addresses rail-structure interaction under seismic events, including performance objectives and design methodology.

3.6.4 Blast Loading

Blast loadings and measures to mitigate are not specified at this time.

3.7 LOAD FACTORS AND LOAD MODIFIERS

For structures carrying high-speed trains, the design shall be in accordance with CBDS Section 1.3.2:

$$\Sigma\ \eta_i\ \gamma_i\ Q_i\ \leq\ \Phi R_n = R_r$$

Where:

γ_i = load factor applied to force effects. (see Tables 3-2, 3-3, and 3-4)

Φ = resistance factor applied to minimal resistance (see Table 3-5)

η_i = load modifier relating to ductility, redundancy and importance (CBDS, section 1.3.2)

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance, ΦR_n

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 100 year design life of the facility. The load modifier is based upon AASHTO LRFD Article 1.3.2, and is applicable to Strength Limit Load Combinations only.

Variation of load factors for permanent loads, $\gamma_{p,}$ are presented in Table 3-3 and 3-4.

Some selected resistance factors “Φ” are presented in Table 3-5.

3.7.1 Design Load Combinations

The load combinations to be used for structures carrying high-speed trains are presented in Table 3-2. The description of the load combinations 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.
- “Extreme 1” is the load combination for derailment.
- “Extreme 2” is the load combination for collision.
- “Extreme 3” is the load combination for seismic events: Maximum Considered Earthquake (MCE), Design Basis Earthquake (DBE), and Lower-level Design Basis Earthquake (LDBE). Each event shall be applied individually, and shall comply with the respective performance criteria.
- ”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 only to tension in prestressed concrete columns with the objective of crack control.
- “Buoyancy at Dewatering Shutoff” is for evaluation of uplift with a minimum weight structure.
- “Fatigue” is the fatigue and fracture load combination relating to repetitive train loading.

Note that for each load combination shown below, all physically achievable subsets (i.e.: omitting load factors by setting γ_i = 0) which may govern design shall be considered.

Note that other load cases for train and track structure interaction are contained within TM 2.10.10: High-Speed Train and Track Structure Compatibility.

Table 3-2 - 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	MCE DBE LDBE 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}	--	--	--
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	--	--	γ_{EQ}	--	--	--	--	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	0.70	--	1.00/ 1.20	--	1.00	--	--	--
Buoyancy @ Dewatering Shutoff	0.80	0.80	1.00	0.45	--	--	--	--	--	--	--
Fatigue	--	1.00	--	--		--	--	--	--	--	--

Where:

- γ_{TG} is equal to 1.0 when live load is not considered, and 0.50 when live load is considered.
- γ_{EQ} is equal to 0.0 for MCE and DBE, and 0.50 for LDBE.
- 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.
- γ_{TU} is equal to the larger value for deformations, and the lesser value for force effects.
- Derailment load factor taken greater than unity to account for absence of dynamic impact.
- WS load factors for Service I and Strength 4 are larger than the CBDS to account for a higher wind speed under train operations. Operation of trains is assumed to cease at a wind speed of 67 mph, which is subject to modification based on future technical memoranda.

Table 3-3 - Load Factors for Permanent Loads, γ_P

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		γ_p Load Factor	
		Maximum	Minimum
DC: Components and Attachments DC: Strength 3 only		1.25 1.50	0.90 0.90
DD: Downdrag	Piles: α Tomlinson Method Piles: λ Method Drilled Shafts: O'Neill and Reese (1999) Method	1.40 1.05 1.25	0.25 0.30 0.35
DW: Non-structural dead load and non-permanent attachments		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest • AEP for Anchored Walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: locked-in construction forces		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structures • Rigid Frames • Flexible Buried Structures other than Metal Box Culverts • Flexible Metal Box Culverts		 1.00 1.35 1.30 1.35 1.95 1.50	 N/A 1.00 0.90 0.90 0.90 0.90
ES: Surcharge Loads		1.50	0.75
SE: Earth Settlement Effects		1.50	0.9

Table 3-4 - Load Factors for Permanent Loads due to Superimposed Deformations, γ_P

Bridge Component	PS	CR, SH
Superstructures - Segmental Concrete Substructures supporting Segmental Superstructures (See CBDS Sections 3.12.4 and 3.12.5)	1.00	see γ_P for DC, Table 3-3
Concrete Superstructures – non-segmental	1.00	1.00
Substructures supporting non-segmental superstructures • Using I_{gross} • Using $I_{effective}$	 0.50 1.00	 0.50 1.00
Steel Substructures	1.00	1.00

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 post-tensioning
- CR: creep effects
- SH: shrinkage effects
- WA: water loads

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
- DBE: Design Basis Earthquake
- LDBE: Lower-lLevel Design Basis Earthquake
- WAD: hydrodynamic force
- ED: dynamic earth pressures

3.7.2 Resistance Factors

Selected resistance factors Φ, with CBDS reference, are listed in Table 3-5.

Table 3-5 Selected Resistance Factors, Φ

Description	Factor	CBDS Reference
Tension controlled reinforced concrete	0.90	5.5.4.2
Tension controlled prestressed concrete	1.00	5.5.4.2
Shear and torsion, normal weight concrete	0.90	5.5.4.2
Compression controlled sections with spirals or ties	0.75	5.5.4.2
Bearing on concrete	0.70	5.5.4.2
Flexure or shear on structural steel	1.00	6.5.4.2
Axial compression in structural steel	0.90	6.5.4.2
Axial tension, yielding of gross section in structural steel	0.95	6.5.4.2
A325 and A490 bolts in shear or tension	0.80	6.5.4.2
Aluminum in tension, ultimate	0.90	7.5.4
Aluminum in tension, yield	0.75	7.5.4
Aluminum in compression	0.80	7.5.4
Drilled shaft tip resistance in clay	0.40	10.5.5
Drilled shaft side resistance in clay	0.45	10.5.5
Drilled shaft tip resistance in sand	0.50	10.5.5
Drilled shaft side resistance in sand	0.55	10.5.5
Driven piles in axial compression in clay (value depends on design method)	0.25 -0.40	10.5.5
Driven piles in axial compression in sand (value depends on design method)	0.30 – 0.45	10.5.5
Tensile resistance or soil anchor tendon, A615 bars	0.90	11.5.6
Tensile resistance or soil anchor tendon, A722 bars	0.80	11.5.6

For design of elements not listed refer to CBDS.

4.0 SUMMARY AND RECOMMENDATIONS

The recommended structural design loads are presented in Section 6.0.

5.0 SOURCE INFORMATION AND REFERENCES

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. ACI: American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318-05
3. AISC: American Institute of Steel Construction, Steel Construction Manual, 13th Edition
4. AWS D1.1/D1.1M:2008 Structural Welding Code-Steel
5. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code
6. AWS D1.8/D1.8M:2009 Structural Welding Code-Seismic Supplement
7. CBC: The 2007 California Building Code
8. California Department of Transportation (Caltrans) Bridge Design Manuals, latest edition
    - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments.
    - Bridge Memo to Designers Manual (CMTD)
    - Bridge Design Practices Manual (CBPD)
    - Bridge Design Aids Manual (CBDA)
    - Bridge Design Details Manual (CBDD)
    - Bridge Memo to Designers Manual (CMTD)
    - Standard Specifications
    - Standard Plans
    - Seismic Design Criteria ver. 1.5 (CSDC)
9. European Standard EN 1991-2:2003 Traffic Loads on Bridges
10. European Standard EN 1990 annex A2: Application to Bridges
11. Taiwan High Speed Rail (THSR) Corporation: Volume 9, Sections 1, 3, and 9
12. Goyal, A. and Chopra, A. K., “Earthquake Analysis and Response of Intake-Outlet Towers”, Report No. EERC 89-04, Earthquake Engineering Research Center, University of California, Berkeley, July 1989.
13. Mononobe-Okabe: See CBDS, Appendix A11
14. CHSTP TM 2.10.4: Interim Seismic Design
15. CHSTP TM 2.9.10: Geotechnical Analysis and Design Criteria Guidelines
16. CHSTP TM 2.10.10: High-Speed Train and Track Structure Compatibility
17. CHSTP TM 2.10.6: Surface Fault Rupture Analysis and Design
18. CHSTP TM 3.2.2: OCS Structural Requirements
19. CHSTP TM 2.5.1: Structural Design of Surface Facilities and Buildings
20. CHSTP TM 2.1.5: Track Design

6.0 DESIGN MANUAL CRITERIA

6.1 GENERAL

This section provides design guidance for load effects required under the Load and Resistance Factor Design (LRFD) methodology for permanent and transient loads for structures directly supporting dedicated high-speed trains for the California High-Speed Train Project (CHSTP). This section also provides guidance for structural classification and highlights design parameters.

Design guidance provided herein defines loads specific to CHSTP bridges, aerial structures, and grade separations. Although this section provides design guidance for loads which could be deemed applicable to earth retaining structures and cut-and-cover structures, it is not currently intended to be comprehensive with regard to these structure types, and shall not be used for their design.

Facility loads, such as those for buildings and stations not supporting high-speed trains, are covered in TM 2.5.1: Structural Design of Surface Facilities and Buildings.

The project-specific high-speed rolling stock has not yet been selected; a Cooper E-50 for the live loading has been selected for preliminary design.

For structures carrying highway loads, Caltrans Bridge Design Specifications (CBDS) shall generally apply, with supplementary provisions herein which account for loads or performance criteria specific to high-speed train operations.

Specific criteria and evaluation measures have not been developed for existing structures which could impact operability of high-speed trains in event of failure. For example, procedures have not been developed for evaluation or assessment of an existing highway bridge which spans over a high-speed trackway. Future criteria will address this.

Requirements specific to train-structure compatibility and rail-structure interaction are addressed in TM 2.10.10: High-Speed Train and Track Structure Compatibility.

6.1.1 General Classifications

CHST facility structures are classified as:

- Bridges – high-speed train trackway structures crossing rivers, lakes or other bodies of water
- Aerial Structures – elevated high-speed train trackway structures
- Grade Separations – structures separating high-speed train trackways from highway or pedestrian usage.
- Earth Retaining Structures – including U-walls and retaining walls
- Cut-and-Cover Underground structures – including cut-and-cover subway line structures
- Bored Tunnel Linings
- Mined Tunnels
- Buildings and All Other Above-ground Structures – 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

CHSTP facilities, based on their importance to high-speed train service, are classified as Primary or Secondary Structures.

Primary Structures: Primary Structures are those that directly support track and running trains, including bridges, aerial structures, stations, tunnels and underground structures, earth retaining structures, and embankments. Primary Structures include facilities essential to train service including, train control, operation, communication, traction power, power distribution network, equipment, and maintenance facilities. Primary Structures also include systems essential to train service including, tracks, rail fasteners, slab track, and ballast.

Secondary Structures: Secondary Structures are those that are not necessary for immediate resumption of train service including, administrative buildings, shop buildings, storage facilities, parking structures and training facilities.

This document is related to design of Primary Structures.

6.1.2 Structural Design Parameters

1. All structures shall be designed for the appropriate loadings and shall comply with the structure gauge adopted for the high-speed train system.
2. Structure design load assumes dedicated high-speed rail operations. Freight rail vehicles will not operate on high-speed rail lines.
3. Structural design guidance shall apply to all structures adjacent to, above, or below the high-speed tracks. This includes structures carrying high-speed trains, and newly constructed highway or ancillary structures, should they directly affect high-speed rail operations. For example, a newly constructed highway bridge above or adjacent to a high-speed rail bridge shall meet CHSTP seismic performance criteria for the MCE event, since its potential failure would directly affect high-speed rail operations.
4. The design life of fixed facilities shall be 100 years. Elements that are normally replaced for maintenance, such as expansion joints, may be designed to a shorter term.
5. The maximum design speed for the main tracks is 250 miles per hour; segments of the alignment will be designed to lesser speeds.
6. The bridges and aerial superstructures shall be designed as essentially rigid and stiff in order to meet serviceability and comfort requirements for high-speed train operation.
7. Design and construction of high-speed train facilities shall comply with the approved and permitted environmental documents.

6.1.3 Seismic Design Parameters

For seismic design guidance and performance, refer to TM 2.10.4: Interim Seismic Design.

Based upon the structure's importance and technical classification, it may need to comply with multiple seismic performance criteria.

TM 2.10.6: Surface Fault Rupture Analysis and Design requires that tracks cross major capable fault zones at-grade, with near perpendicular alignment to the fault trace.

Project specific seismic criteria are pending oversight by a Technical Advisory Panel (TAP).

6.2 DESIGN CODES AND SPECIFICATIONS

The structural design shall meet all applicable portions of the general laws and regulations of the State of California and of respective local authorities. The design of the CHST structures use Load and Resistance Factor Design (LRFD) methodology.

Unless otherwise specified, the CHSTP facilities shall be designed in accordance with applicable portions of the following standards and codes:

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. ACI: American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318-05
3. AISC: American Institute of Steel Construction, Steel Construction Manual, 13th Edition
4. AWS D1.1/D1.1M:2008 Structural Welding Code-Steel
5. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code
6. AWS D1.8/D1.8M:2009 Structural Welding Code-Seismic Supplement

7. CBC: The 2007 California Building Code
8. California Department of Transportation (Caltrans) Bridge Design Manuals, latest edition
    - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments.
    - Bridge Memo to Designers Manual (CMTD)
    - Bridge Design Practices Manual (CBPD)
    - Bridge Design Aids Manual (CBDA)
    - Bridge Design Details Manual (CBDD)
    - Bridge Memo to Designers Manual (CMTD)
    - Standard Specifications
    - Standard Plans
    - Seismic Design Criteria ver. 1.5 (CSDC)

In addition, the design of structures to be built as part of the CHSTP but owned by other agencies or private owners, shall meet the requirements of the agencies which normally have jurisdiction over such. In the case of differing values, the standard followed shall be that which results in the satisfaction of all applicable requirements. In the case of conflicts, documentation for the conflicting standard is to be prepared and approval is to be secured as required by the affected agency for which an exception is required, whether it be an exception to the CHSTP standards or another agency's standards.

6.3 PERMANENT LOADS

6.3.1 Dead Load (DC, DW)

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

DC refers to the dead load of structural components and permanent attachments supported by the structure including, tracks, ballast, plinths, walkways, sound walls, overhead contact system, etc.

DW refers to the dead load of non-structural attachments which may be non-permanent including, wearing surfaces, utilities, cable trays, finishes, etc. If applicable, dead load shall be applied in stages to represent the sequence required to construct the structure. Analysis shall include consideration of the maximum and minimum loading that may be imposed on the structure either during construction or that resulting from future placement or removal of the earth cover.

In the absence of more precise information, units weights specified in Table 6.3-1 shall be used to calculate dead load:

Table 6.3-1 – Unit Weights of Common Materials

Item		Unit Weight
Aluminum alloys		175 pcf
Ballast, Rolled Gravel, Macadam		140 pcf
Bituminous Wearing Surfaces		140 pcf
Cast Iron		450 pcf
Cinder Filling		60 pcf
Compacted Sand, Silt, or Clay		120 pcf
Unreinforced Concrete	Lightweight	110 pcf
	Sand-Lightweight	120 pcf
	Normal Weight with f'c ≤ 5 ksi	145 pcf
	Normal Weight with 5 < f'c ≤ 15 ksi	(140 + f'c) pcf
Electrification (overhead system and fastenings)		100 pounds per foot of track
Loose Sand, Silt, Gravel or Soft Clay		100 pcf
OCS poles/precast duct bank including walkway surface and walkway barrier		300 psf
OCS wire break		13.3 kip in direction of intact wire each track
Rails and fasteners (no ties)		200 pounds per foot of track
Slab track & slab track base		10 inches of concrete between OCS poles or curbs. This will be adjusted if a proprietary slab track is selected.
Soils		See Geotechnical recommendations
Steel		490 pcf
Stone Masonry		170 pcf
Systems cable tray		180 pounds per foot of track
Water	Fresh	62.4 pcf
	Salt	64.0 pcf
Wood	Hard	60 pcf
	Soft	50 pcf

6.3.2 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 Design Report.

6.3.3 Earth Pressures (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 Design Report.

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 Design Report shall be used.

Lateral Static Earth Pressure (EH): For structures retaining draining cohesionless (granular) soil, lateral earth pressure shall be determined in accordance with the following paragraphs of these criteria. For structures retaining other soil types, the definitions shall be provided for those soil types and shall be included in the Geotechnical Design Report.

Yielding Walls: Yielding walls (i.e.: typical cantilever retaining walls) are defined as walls which are unrestrained and free to move at the top a distance of at least 0.004H, where H is defined as the height of the wall from the base of the heel to the finished grade directly above the heel.

For yielding walls, the lateral static earth pressure shall be determined using the active lateral pressures expressed as equivalent fluid pressures. Recommended values given in the Geotechnical Design Report shall be used.

Rigid Walls: Rigid walls are defined as walls which are restrained at the top so that the amount of deflection required to develop active pressure conditions is not possible. All permanent excavation support walls as well as tunnel portal transitions and U-sections shall be considered rigid walls.

For rigid walls, the static lateral soil pressure shall be determined using the at-rest lateral pressures expressed as equivalent fluid pressures. Recommended values given in the Geotechnical Design Report shall be used.

6.3.4 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 recommended in the Geotechnical Design Report.

Underground structures and walls shall be designed for loading from existing adjacent buildings or structures. Consideration shall be given to the maximum and minimum loads that can be transferred to the design structure, and design loads shall be assumed to be the same as those for which the adjacent structure was designed. In the absence of this information, loads based on provisions in the CBC concurrent with the heaviest occupancy for which the building is suitable, shall be used.

Surcharge loads from adjacent buildings and structures commonly result from combinations of dead load, live load, etc. Due to this, surcharge loads applied for the design structure need not be factored twice, and shall be taken as the greater of the unfactored surcharge loading multiplied by the specified factor, ES, or the factored surcharge loading with ES equal to 1.0.

Structures that may support future developments above CHSTP facilities shall be designed to reflect the future load. Surcharge loads modeling future developments shall not be applied concurrently with traffic or future traffic surcharges in the same locations. If a future development extends to the surface land adjacent to an underground structure, then a surcharge shall be applied for design. The surcharge shall model the loading due to the maximum development currently allowed by zoning.

Construction activities may result in permanent or temporary loads from equipment, trucks, drayage, stockpiling of materials, or excavated earth. A surcharge shall be applied to all underground structures, unless: 1) positive and recognizable means are provided at the ground surface to ensure that the above types of loadings cannot occur; and, 2) the Authority specifically permits in writing, the application of a surcharge of lesser magnitude.

The vertical surcharge shall be considered as a static uniform load applied as follows:

600 psf for h < 5ft

600-40 (h-5) psf for 5ft ≤ h ≤ 20ft

0 psf, for h > 20ft

h = vertical distance from the top of structure to ground surface, in feet.

Surcharge loads representing construction activities need not be applied concurrently with traffic or future traffic surcharges in the same location.

See Article 6.4.1.5 for Live Load Surcharge (LLS).

6.3.5 Earth Settlement Effects (SE)

Earth settlement effects (SE) are forces or displacements imposed on a structure due either uniform or differential settlement under sustained loading. Recommended values of settlement as given in the Geotechnical Design Report shall be used. Tolerable settlement on foundations shall be developed consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on structural and operational performance.

Specific settlement criteria may be revised/developed at a later date.

6.3.6 Creep Effects (CR)

For the effects due to creep of concrete (CR), the requirements in CBDS Section 5.4.2.3 shall be used.

6.3.7 Shrinkage Effects (SH)

For the effects due to shrinkage of concrete (SH), the requirements in CBDS Section 5.4.2.3 shall be used.

6.3.8 Secondary Forces from Prestressing (PS)

Secondary forces from prestressing (PS) shall be considered. Such secondary forces arise during prestress of statically indeterminate structures, which produce internal forces and support reactions. Secondary forces are generally obtained by subtracting the primary prestress forces from the total prestressing.

6.3.9 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.

6.3.10 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 CBDS Section 3.7. Recommended values given in the Geotechnical Design Report shall be used.

Adequate resistance to flotation shall be provided at all sections for full uplift pressure on the structure foundation, based upon the maximum probable height of the water table defined in the Geotechnical Design Report. For the completed structure, such 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 all joints). Friction acting on rigid excavation support systems such as diaphragm walls in permanent structures may also be used to resist uplift.

Hydrostatic pressure shall normally be applied to all surfaces in contact with groundwater with a magnitude based on the depth of water and the applicable water density.

The change in foundation condition due to scour shall be investigated per CBDS Article 3.7.5.

6.4 TRANSIENT LOADS

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

Live loads are due to high-speed trains, Cooper E-50 maintenance trains, highway loads, construction equipment, and pedestrians.

6.4.1.1 Floor, Roof, and Pedestrian Live Loads (LLP)

For the force effects due to floor and roof live loads (LLP), reference TM 2.5.1: Structural Design of Surface Facilities and Buildings. Technical Memorandum 2.5.1 includes provisions for aerial trackway supporting service walkways.

6.4.1.2 High-Speed Train Loads (LLV)

The project specific rolling stock has not yet been determined. Once the project specific rolling stock is determined, the live load criteria will be expanded to consider these design trains.

Where a structure supports multiple tracks, the loading shall be applied for those number of tracks either simultaneously or individually, whichever governs design.

6.4.1.3 Maintenance and Construction Train Live Loads (LLRR)

Structures shall be designed to support maintenance and construction trains, defined as the Cooper E-50 in the AREMA Specification, see Figure 6-1.

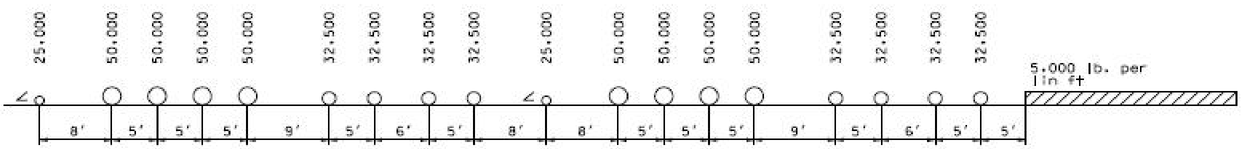


Figure 6-1: Cooper E-50 (LLRR)

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.

6.4.1.4 Highway Live Loads (LLH)

Facilities designed to support highway loads shall be designed for a minimum of HL-93 Truck Loading based on CBDS Article 3.6.1. For facilities that support highway permit loads, Caltrans guidelines shall be followed for the routing and sizes of the permit vehicles.

6.4.1.5 Live Load Surcharge (LLS)

An area 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 may result 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 CBDS, Article 3.6.

No impact factors apply to LLS for walls. A reduction of impact for buried components shall be applicable as specified in CBDS Article 3.6, with the 33% 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 Design Report.

6.4.1.6 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 all structures.

Fatigue assessment criteria is pending the next revision of this technical memorandum.

6.4.2 Vertical Impact Effect (I)

Moving trains and vehicles impart dynamic load 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 high-speed trains (LLV) in order to determine impact effects. This is addressed in detail within TM 2.10.10: High-Speed Train and Track Structure Compatibility.

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:

$$I = \frac{225}{\sqrt{L}}$$ where 14 ft < L ≤ 127 ft

$$I = 20\%$$ where L > 127 ft.

Steel bridges:

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

$$I = 16 + \frac{600}{L - 30}$$ where L ≥ 80 ft

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

Direct fixation on concrete slab track:

$$I = 100\left(\frac{2.16}{\sqrt{0.305L} - 0.2} - 0.27\right) \le 100\%$$ where L ≤ 40 ft

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

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

An additional 20% 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), CBDS shall be used.

Impact 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 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)

6.4.3 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.

For centrifugal forces carrying vehicular traffic refer to CBDS.

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

For LLRR, CF acts at 8 feet above top of rail

For LLV, CF acts at 6 feet above top of rail

<u>For U.S. customary units</u>:

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

V = train speed (mph)

R = horizontal radius of curvature (ft)

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

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

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

$f = 1$, for LLV, all speeds

L_f = length in feet (ft) 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 cant shall be considered when present. The cant 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.

6.4.4 Traction and Braking Forces (LF)

Traction and braking forces (LF) act at the top of the rails in the longitudinal direction of the track.

For traction and braking forces from maintenance and construction trains (LLRR) use AREMA. For braking forces from truck loading (LLH) refer to CBDS.

For traction and braking forces from high-speed trains acting at top of rail (LLV):

Traction force = 2.26L (kips), or 25% of LLV < 225 kips

Braking force = 1.37L (kips), or 25% of LLV < 1350 kips

where L = length in feet of portion of bridge under consideration

These loads are to be distributed equally to all axles over the length of the train.

Note that traction and braking forces will be reviewed and possibly revised once the project-specific rolling stock is selected.

6.4.5 Nosing and Hunting Effects (NE)

For structures carrying high-speed trains with slab track and direct fixation fasteners, nosing and hunting effects (NE) of the wheels contacting the rails shall be accounted by a 22kip force applied to the top of the low rail, perpendicular to the track centerline 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 (CE).

6.4.6 Wind Loads (WS, WL)

Wind Pressure on Structures (WS) and Wind Pressure on Trains (WL) shall be calculated in accordance with requirements in CBDS Section 3.8 with the following modifications:

(1) 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 all parapets of significant height contained on the bridge).

(2) Use of "Suburban" and "City" exposure categories to calculate design wind velocity is discouraged and shall be accepted only on a case-by-case basis.

(3) The base lateral load for Wind Pressure on Vehicles (WL) shall be revised to 0.300 klf perpendicular to the train acting 8 feet above the top of rail. CBDS 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.

(4) For structures which utilize soundwalls or windwalls 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 which is shielded by the wall, assuming the train to be 14 feet tall with a lower border at 1 foot from top of rail. However this reduction shall not exceed 50% of WL.

Local design elements such as parapets or components on structures shall give consideration to wind loading and slipstream effects. Wind loading on these elements may be calculated per applicable building code, as detailed in TM 2.5.1: Structural Design of Surface Facilities and Buildings. 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 TM 2.5.1.

Wind loads (WS, WL) on highway structures shall be per CBDS.

6.4.7 Slipstream Effects (SS)

<u>LLV</u>

6.4.7.1 Aerodynamic Actions from Passing Trains

The passing of high-speed trains subjects any structure situated near the track to travelling waves of alternating pressure and suction. 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 travelling wave of alternating pressure and suction (see Figures 6-2 to 6-5). The magnitude of the action depends mainly on:
  - square of the speed of the train
  - aerodynamic shape of the train
  - shape of the structure

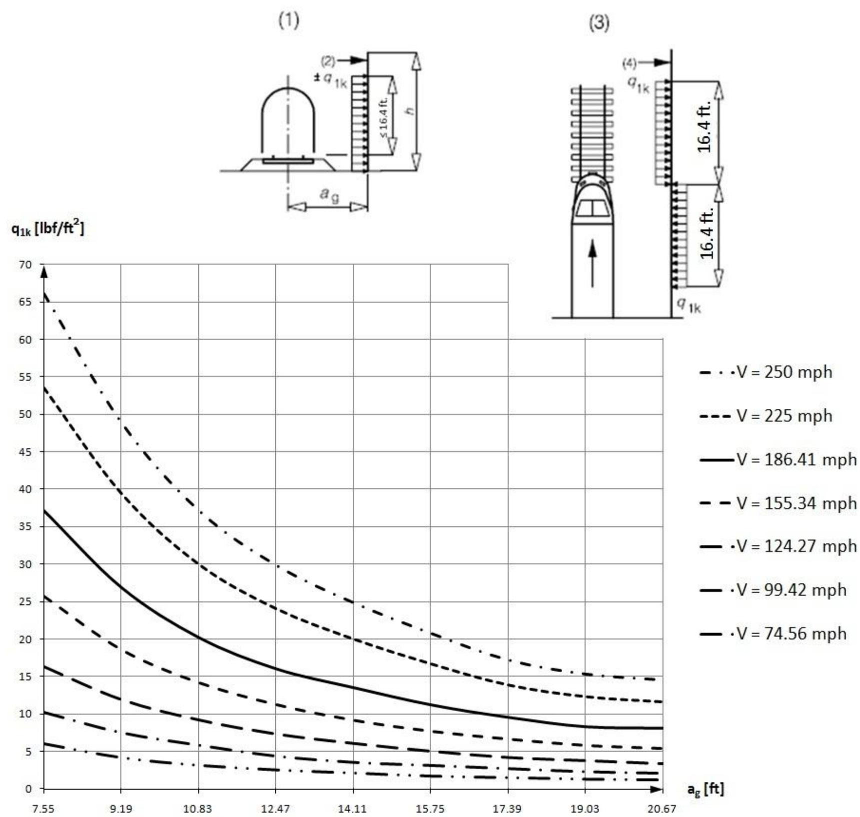
- position of the structure, particularly the clearance between the vehicle and the structure

- The actions may be approximated by equivalent loads at the head and rear ends of a train, when checking ultimate and serviceability limit states and fatigue. Equivalent loads are given in Sections 0 to 0.
- In Sections 0 to 0 the Maximum Design Speed V [mph] should 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 16.5 feet from the start and end of the structure measured parallel to the tracks the equivalent loads in Sections 0 to 0 should be multiplied by a dynamic amplification factor of 2.0.

NOTE: For dynamically sensitive structures the above dynamic amplification factor may be insufficient and may need to be determined by a special study. The study should take into account dynamic characteristics of the structure including support and end conditions, the 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.

6.4.7.2 Simple Vertical Surfaces Parallel to the Track (e.g. noise barriers)

(1) Equivalent loads, ± q_{1k}, are given in Figure 6-2



Key
(1) Section
(2) Surface of structure
(3) Plan view
(4) Surface of structure

Figure 6-2: Equivalent loads q_{1k} for simple vertical surfaces parallel to the track

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

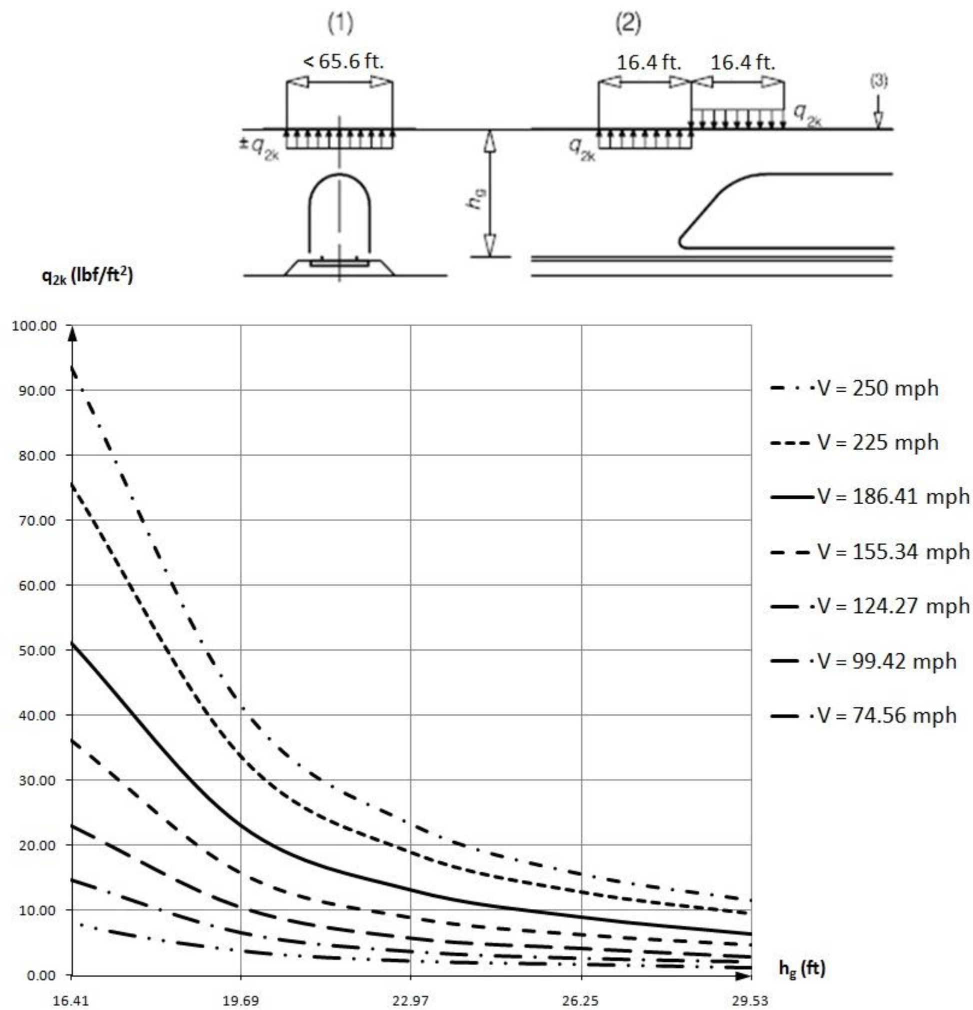
- a factor k_1 = 0.85 for trains with smooth sided rolling stock
- a factor k_1 = 0.6 for streamlined rolling stock (*e.g.* ETR, ICE, TGV, Eurostar or similar)

(3) If a small part of a wall with a height < 3.00 feet and a length < 8 feet is considered, *e.g.* an element of a noise protection wall, the actions q_{1k} should be increased by a factor k_2 = 1.3.

6.4.7.3 Simple Horizontal Surfaces above the Track (e.g. overhead protective structures)

(1) Equivalent loads, ± q_{2k}, are given in Figure 6-3.

(2) The loaded width for the structural member under investigation extends up to 33 feet to either side from the centerline of the track.



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

Figure 6-3: Equivalent loads q_{2k} for simple horizontal surfaces above the track

(3) For trains passing each other in opposite directions the actions should be added. The loading from trains on only two tracks needs to be considered.

(4) The actions q_{2k} may be reduced by the factor k_1 as defined in Section 0.

(5) The actions acting on the edge strips of a wide structure crossing the track may be multiplied by a factor of 0.75 over a width up to 5 feet.

6.4.7.4 Simple Horizontal Surfaces Adjacent to the Track (e.g. platform canopies with no vertical wall)

(1) Equivalent loads, $\pm q_{3k}$, are given in Figure 6-4 and apply irrespective of the aerodynamic shape of the train.

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

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

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

$$k_3 = 0 \quad \text{for } h_g \geq 24.6 \text{ ft}$$

where:

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

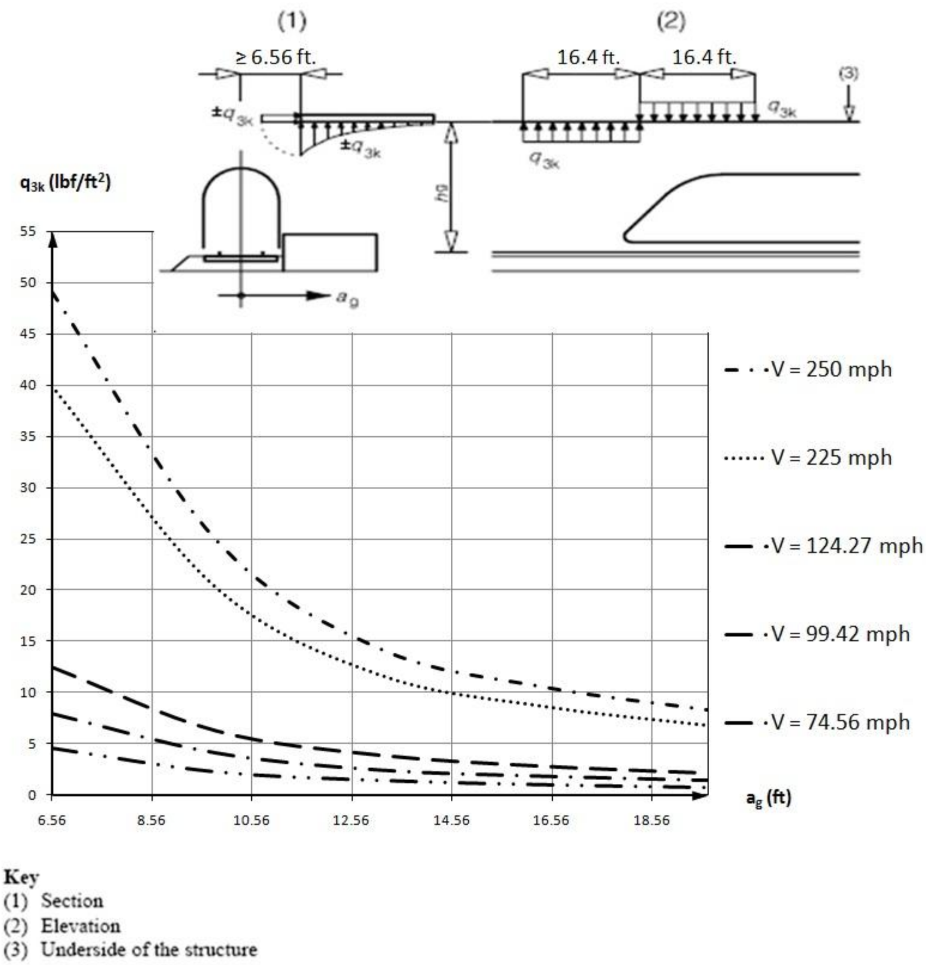


Figure 6-4: Equivalent loads q_{3k} for simple horizontal surfaces adjacent to the track

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

(1) Equivalent loads, ± q_{4k}, as given in Figure 6-5 should be applied normal to the surfaces considered. The actions should be taken from the graphs in Figure 6-2 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 6-5

(2) If max a_g > 20 feet the value max a_g = 20 feet should be used

(3) The factors k_1 and k_2 defined in Section 0 should be used

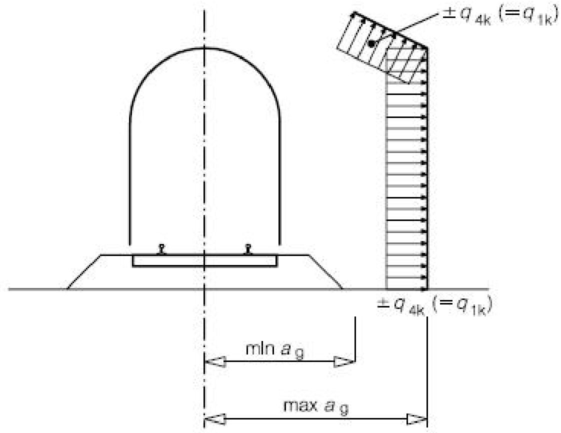


Figure 6-5: Definition of the distances min a_g and max a_g from center-line of the track

6.4.7.6 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)

(1) All actions should 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 0

  $k_4 = 2$

- to the horizontal surfaces:

  $\pm k_5 q_{2k}$

  where:

  q_{2k} is determined according to Section 0 for only one track,

  k_5 = 2.5 if one track is enclosed,

  k_5 = 3.5 if two tracks are enclosed

Additional analyses needs to be developed and tested for final design.

6.4.8 Thermal Load (TU,TG)

For uniform (TU) and gradient (TG) temperature effects of the structure, the requirements in CBDS Section 3.12 shall be used.

For rail thermal ranges, including rail setting temperatures, reference TM 2.1.5: Track Design.

6.4.9 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 CBDS shall be used.

6.4.10 Seismic Loads (MCE, DBE, LDBE)

Detailed, project specific seismic design criteria are presented in Tech Memo 2.10.4: Interim Seismic Design Criteria. TM 2.10.4 defines seismic design philosophies, seismic analysis/demand methodologies, and structural capacity evaluation procedures for the multiple levels of design earthquakes.

In general an individual structure may need to comply with multiple performance levels, depending upon its Importance and Technical Classification.

Highway bridges spanning over high-speed train trackways, or highway structures which have the capacity to influence operability of high-speed trains in event of a failure, shall be designed for the Maximum Considered Earthquake (MCE) seismic event per provisions in TM 2.10.4: Interim Seismic Criteria, in addition to Caltrans Seismic Design Criteria (CSDC).

6.4.11 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 [12] or by equivalent means.

For possible additional hydrodynamic force effects, see the Geotechnical Design Report.

6.4.12 Dynamic Earth Pressures (ED)

Dynamic earth pressure due to seismic motion on retaining structures shall be computed using the methods by Mononobe-Okabe analysis [13].

For possible additional dynamic earth pressure, see the Geotechnical Design Report.

6.4.13 Derailment Loads (DR)

6.4.13.1 LLRR and LLV

In the event of a derailment, the damage to the structures carrying high-speed trains shall be minimal. Overturning or collapse of the structure shall not be allowed, but local damage can be tolerated.

Two 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 an upstand 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 nonstructural elements such as walkways).

For Case I, collapse of a major part of the structure shall be avoided. Local damage, however, may be tolerated. The parts of the structure concerned shall be designed for the following design loads in the Extreme Loading Combination:

- Cooper E-50 loading (both point loads and uniformly distributed loading) 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 centerline of the track, or as limited by containment walls. If a short 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 short wall.

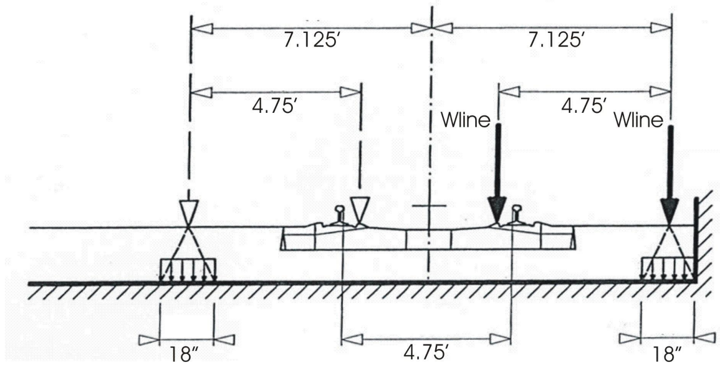


Figure 6-6: Derailment Case I

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

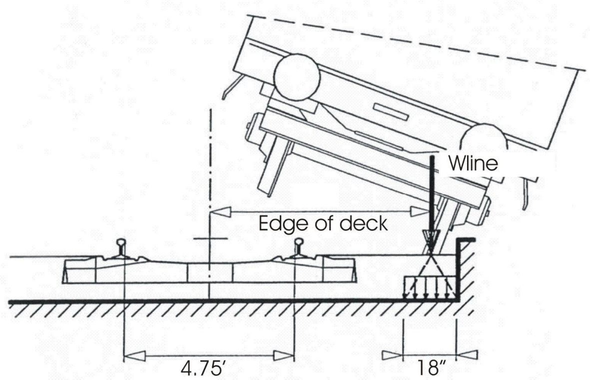


Figure 6-7: Derailment Case II

Design Situations 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 Figures 6-6 and 6-7.

For Design Situations I and II, other rail traffic actions should be neglected for the track subjected to derailment actions. When the structure under consideration carries more than one track, only one 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 apply in this case.

No dynamic factor needs to be applied to the derailment loads; however the loads shall be multiplied by the load factor within load combinations. A load factor of 1.0 shall apply to the horizontal force and a load factor of 1.4 shall apply to the vertical component.

For structural elements which are situated above the level of the rails, measures to mitigate the consequences of a derailment shall be in accordance with the specified requirements in addition to the above.

6.4.13.2 Track Side Containment

Derailment containment walls shall be provided for all mainline aerial structures at locations 7 feet or less from the track centerline. The height of the wall shall be minimum 0.67 feet above the level of the adjacent track's lower rail. A transverse horizontal concentrated load of 35 kips shall be applied at top of the wall.

6.4.14 Collision Loads (CL)

Collision loads in Sections **Error! Reference source not found.**6.4.14.1 to 6.4.14.2 apply to train impact loads (LLRR, LLV). Section 6.4.14.30 applies to highway collision loads (LLH). For collision loads on columns or divider wall along the trackway, reference TM 2.5.1: Structural Design of Surface Facilities and Buildings.

6.4.14.1 Collision Loads other than at Stations or Platforms

Members within 16.5 feet of the track center line shall be designed to resist train collision, 900 kips parallel to track, or 350 kips perpendicular to tracks.

6.4.14.2 Structures in Areas beyond Track Ends

(1) Overrunning of rail traffic beyond the end of a track or tracks (for example at a terminal station) should 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.

(2) The measures to manage the risk should 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.

(3) Supporting structural members to structures should generally not be located in the area immediately beyond the track ends.

(4) Where supporting structural members are required to be located near to track ends, an end impact wall should be provided in the area immediately beyond 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 engines. It is recommended that these forces are applied horizontally and at a level of 3 feet above track level.

6.4.14.3 Highway Vehicle Collision Loads (LLH)

Highway collision loading shall be as per CBDS Section 3.6.5.

6.5 Miscellaneous Loads

6.5.1 Overhead Contact System (OCS) Loads

Design of OCS poles and foundations shall be in accordance with TM 3.2.2: OCS Structural Requirements.

Loads shall include those arising from normal service, dead load, wind load, slipstream effects, seismic load, and unbalanced loads due to wire installation or wire breakage.

Deflection requirements shall be met as outlined in TM 3.2.2.

Catenary pole foundations shall be capacity-protected to prevent failure or unrecoverable damage to the bridge deck.

Aerial trackway structures shall be designed in such a manner that the OCS pole foundation locations can be determined at a later time.

6.5.2 Construction Loads and Temporary Structures

6.5.2.1 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 high-speed trains and/or pass over routes carrying high-speed trains. Subsequent articles herein apply to Type A structures.

- Type B: Temporary structures or structures under temporary conditions which do not carry high-speed trains and do not pass over routes carrying high-speed trains. These structures shall be designed in accordance with the specifications of the owning/operating agency (*e.g.* Caltrans: CBDS and CMTD). Structures such as haul bridges used temporarily for the CHSRP shall be designed in accordance with CMTD 15-14.

6.5.2.2 Construction Load Combinations

Temporary structures or structures under temporary conditions shall be designed to adequately resist conditions at all stages of construction, including all applicable construction loads. Construction load combinations 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%.
- Service 1, as applicable.
- Seismic load combinations: For 30% design purposes, Load Combination Extreme 3, with revised ground motions having 10% probability of exceedance in 10 years. Performance criteria shall comply with the Safety Performance Level (SPL) for the Design Basis Earthquake (DBE) for an important, standard structure, see TM 2.10.4: Interim Seismic Design Criteria. If significant construction loads are anticipated or specialized equipment present, this mass shall be accounted for in the seismic design.

In the absence of better criteria, a construction live load of 10 psf shall be assumed on the bridge deck.

6.5.2.3 Segmental Construction and Specialized Equipment

Construction load combinations per CBDS Article 5.14.2 “Segmental Construction” shall be considered. The temporary seismic load event as detailed herein shall be added to the construction load combination at strength limit states; however a 1.0 load factor may 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.

6.5.3 Rail-Structure Interaction Forces

Effects of continuous welded rail (CWR) interaction with the structure through its attachment shall be considered. Design guidance for this interaction is provided in Technical Memorandum 2.10.10: High-Speed Train and Track Structure Compatibility.

Interaction effects of the CWR and structure result after connection of the rail to the structure. These occur in large part due to the following: temperature variations in the rail and structure, braking/traction, creep and shrinkage, and seismic events. Load combinations presented in this document assume this interaction implicit to each load within the combination, with associated load factors.

Structure thermal ranges shall be calculated per provisions in this document. For specific rail information, including thermal ranges, setting temperature, and rail section, reference TM 2.1.5: Track Design.

TM 2.10.10: High-Speed Train and Track Structure Compatibility also address rail-structure interaction under seismic events, including performance objectives and design methodology.

6.5.4 Blast Loading

Blast loadings and measures to mitigate are not specified at this time.

6.6 LOAD FACTORS AND LOAD MODIFIERS

For structures carrying high-speed trains, the design shall be in accordance with CBDS Section 1.3.2:

$$\Sigma\ \eta_i\ \gamma_i\ Q_i\ \leq\ \Phi R_n = R_r$$

Where:

γ_i = load factor applied to force effects. (see Table 6.6-1 to 6.6-3)

Φ = resistance factor applied to minimal resistance (see Table 6.6-4)

η_i = load modifier relating to ductility, redundancy and importance (CBDS, Article 1.3.2)

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance, ΦR_n

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 100 year design life of the facility. The load modified is applicable to Strength Limit Load Combinations only.

Variation of load factors for permanent loads, γ_p, are presented in Table 6.6-2 and 6.6-3.

Some selected resistance factors "Φ" are presented in Table 6.6-4.

6.6.1 Design Load Combinations

The load combinations to be used for structures carrying high-speed trains are presented in Table 6.6-1. The description of the load combinations 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.
- "Extreme 1" is the load combination for derailment.
- "Extreme 2" is the load combination for collision.
- "Extreme 3" is the load combination for seismic events: Maximum Considered Earthquake (MCE), Design Basis Earthquake (DBE), and Lower-level Design Basis Earthquake (LDBE). Each event shall be applied individually, and shall comply with the respective performance criteria.
- "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 only to tension in prestressed concrete columns with the objective of crack control.
- "Buoyancy at Dewatering Shutoff" is for evaluation of uplift with a minimum weight structure.
- "Fatigue" is the fatigue and fracture load combination relating to repetitive train loading.

Note that for each load combination shown below, all physically achievable subsets (i.e.: omitting load factors by setting γ_i = 0) which may govern design shall be considered.

Note that other load cases for train and track structure interaction are contained within TM 2.10.10: High-Speed Train and Track Structure Compatibility.

Table 6.6-1 - 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 LLHR LLH + I LLHL LLHT LF NE CF SS	WA FR	WS	WL	TU	TG	SE	DR	CL	MCE DBE LDBE 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}	--	--	--
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	--	--	γ_{EQ}	--	--	--	--	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	0.70	--	1.00/ 1.20	--	1.00	--	--	--
Buoyancy @ Dewatering Shutoff	0.80	0.80	1.00	0.45	--	--	--	--	--	--	--
Fatigue	--	1.00	--	--		--	--	--	--	--	--

Where:

- γ_{TG} is equal to 1.0 when live load is not considered, and 0.50 when live load is considered
- γ_{EQ} is equal to 0.0 for MCE and DBE, and 0.50 for LDBE
- 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
- γ_{TU} is equal to the larger value for deformations, and the lesser value for force effects
- Derailment load factor taken greater than unity to account for absence of dynamic impact

Table 6.6-2 - 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 DC: Strength 3 only		1.25 1.50	0.90 0.90
DD: Downdrag	Piles: α Tomlinson Method Piles: λ Method Drilled Shafts: O'Neill and Reese (1999) Method	1.40 1.05 1.25	0.25 0.30 0.35
DW: Non-structural dead load and non-permanent attachments		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest • AEP for Anchored Walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: locked-in construction forces		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structures • Rigid Frames • Flexible Buried Structures other than Metal Box Culverts • Flexible Metal Box Culverts		 1.00 1.35 1.30 1.35 1.95 1.50	 N/A 1.00 0.90 0.90 0.90 0.90
ES: Surcharge Loads		1.50	0.75
SE: Earth Settlement Effects		1.50	0.9

Table 6.6-3 - Load Factors for Permanent Loads due to Superimposed Deformations, γ_P

Bridge Component	PS	CR, SH
Superstructures - Segmental Concrete Substructures supporting Segmental Superstructures (See CBDS Sections 3.12.4 and 3.12.5)	1.00	see γ_P for DC, Table 6.6-2
Concrete Superstructures – non-segmental	1.00	1.00
Substructures supporting non-segmental superstructures • Using I_{gross} • Using $I_{effective}$	 0.50 1.00	 0.50 1.00
Steel Substructures	1.00	1.00

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

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
DBE: Design Basis Earthquake
LDBE: Lower-Level Design Basis Earthquake
WAD: hydrodynamic force
ED: dynamic earth pressures

6.6.2 Resistance Factors

Selected resistance factors Φ, with CBDS reference, are listed in Table 6.6-4.

Table 6.6-4 Selected Resistance Factors, Φ

Description	Factor	CBDS Reference
Tension controlled reinforced concrete	0.90	5.5.4.2
Tension controlled prestressed concrete	1.00	5.5.4.2
Shear and torsion, normal weight concrete	0.90	5.5.4.2
Compression controlled sections with spirals or ties	0.75	5.5.4.2
Bearing on concrete	0.70	5.5.4.2
Flexure or shear on structural steel	1.00	6.5.4.2
Axial compression in structural steel	0.90	6.5.4.2
Axial tension, yielding of gross section in structural steel	0.95	6.5.4.2
A325 and A490 bolts in shear or tension	0.80	6.5.4.2
Aluminum in tension, ultimate	0.90	7.5.4
Aluminum in tension, yield	0.75	7.5.4
Aluminum in compression	0.80	7.5.4
Drilled shaft tip resistance in clay	0.40	10.5.5
Drilled shaft side resistance in clay	0.45	10.5.5
Drilled shaft tip resistance in sand	0.50	10.5.5
Drilled shaft side resistance in sand	0.55	10.5.5
Driven piles in axial compression in clay (value depends on design method)	0.25 -0.40	10.5.5
Driven piles in axial compression in sand (value depends on design method)	0.30 – 0.45	10.5.5
Tensile resistance or soil anchor tendon, A615 bars	0.90	11.5.6
Tensile resistance or soil anchor tendon, A722 bars	0.80	11.5.6

For design of elements not listed refer to CBDS.