California High-Speed Train Project

TECHNICAL MEMORANDUM

Track-Structure Interaction
TM 2.10.10

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Released by:	*Signed document on file* Anthony Daniels, Program Director	09 July 10 Date

Revision	Date	Description
0	30 June 10	Initial Release, R0

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	 Date
Infrastructure:	*Signed document on file* Vince Jacob	30 Jun 10 Date
Operations:	NOT REQUIRED Print Name	 Date
Maintenance:	NOT REQUIRED Print Name	 Date
Rolling Stock:	NOT REQUIRED Print Name	 Date

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 establishes specific requirements for high-speed track and structure interaction for aerial structures and bridges for the California High-Speed Train Project (CHSTP). These requirements encompass tracks supported by either ballast or direct fixation fasteners. The requirements include dynamic performance, traffic safety, rail-structure interaction, and passenger comfort.

These requirements concern limiting aerial structure and bridge deformations and vibrations, which can be magnified under high-speed loading. Excessive deformations and vibrations can lead to numerous issues, including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, ballast instability, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

There are no current Federal Railway Administration (FRA) rules for high-speed (220 mph+) track-structure interaction design. The Federal Railway Administration (FRA) has recently published proposed rules [5] addressing aspects of high-speed track-structure interaction for Class 9 (≤ 220 mph) track. The CHSTP will review and revise this guidance in this technical memorandum as necessary once the final rule amending Track Safety Standards is published by the FRA.

Preliminary track design philosophy per TM 2.1.5: Track Design [9] is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 feet.

This technical memorandum is not intended for tracks supported on grade. Technical memoranda for track supported on grade are pending.

Design level analysis requirements are given for preliminary and final design.

These requirements are necessary on a system-wide basis to ensure that performance requirements of high-speed train structures are met, provide a consistent basis for advancing design, and provide designers a common basis in proportioning materials to allow uniform description of construction techniques and cost estimates.

1.0 INTRODUCTION

1.1 PURPOSE OF TECHNICAL MEMORANDUM

This technical memorandum establishes specific requirements for high-speed track and structure interaction for aerial structures and bridge for the California High-Speed Train Project. These requirements encompass tracks supported by either ballast or direct fixation fasteners. The requirements include dynamic performance, traffic safety, rail-structure interaction, and passenger comfort.

This technical memorandum is not intended for tracks supported on grade. Technical memoranda for track supported on grade are pending.

1.2 STATEMENT OF TECHNICAL ISSUE

Establishing specific requirements for dynamic performance, traffic safety, rail-structure interaction, and passenger comfort of bridges and aerial structures under high-speed trains will provide a consistent basis for design, and result in a system wide criterion applicable to high speed trains.

1.3 GENERAL INFORMATION

1.3.1 Definition of Terms

The following technical terms and acronyms used in this document have specific connotations with regard to the California High-Speed Train system.

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
CDC	CHSTP Design Criteria
CHST	California High-Speed Train
CHSTP	California High-Speed Train Project
DR	Derailment load from high-speed trains
EC or EN	EuroCode
FRA	Federal Railway Administration
I	Vertical impact effect
LDBE	Lower-level Design Basis Earthquake
LF	Acceleration or braking force applied to structures
LLRM	Modified Cooper E-50 loading
LLRR	Maintenance and construction train (Cooper E-50)
LLV	Actual high-speed train
NE	Nosing and hunting effect from trains
PCF	Pounds per cubic foot
PSF	Pounds per square foot
PMT	Program Management Team
THSRC	Taiwan High Speed Rail Corporation

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 establishes specific requirements for high-speed track and structure interaction for aerial structures and bridge for the California High-Speed Train Project (CHSTP). These requirements encompass tracks supported by either ballast or direct fixation fasteners. The requirements include dynamic performance, traffic safety, rail-structure interaction, and passenger comfort.

This technical memorandum is not intended for tracks supported on grade. Technical memoranda for track supported on grade are pending.

This Technical Memorandum shall be used in conjunction with TM 2.3.2: Structure Design Loads.

2.1.1 CHSTP Design Considerations

The Federal Railroad Administration (FRA) issued a notice of proposed rulemaking (Federal Register Vol. 75, No. 89) to revise Title 49 – Transportation, of the Code of Federal Regulations (CFR); Part 213 - Track Safety Standards, and Part 238 – Passenger Equipment Safety Standards [5]. The proposed rule is entitled “Vehicle/Track Interaction Safety Standards; High-Speed and High Cant Deficiency Operations; Proposed Rule”, and creates Track Safety Standards applicable to high-speed and high cant deficiency train operations in order to promote the safe interaction of rail vehicles with the track over which they operate.

The FRA proposed rule sets limits for track perturbations for the range of vehicles currently used and may likely be used on future high-speed or high cant deficiency rail operations, and is based upon results of simulation studies designed to identify track geometry irregularities associated with unsafe wheel/rail forces and accelerations, thorough review of vehicle qualification and revenue service test data, and consideration of international practices. Different classes of track are identified based upon maximum allowable operating speed for the train; the highest of which is Class 9 track for operating speeds up to 220 mph.

The CHSTP will review and revise the guidance in this technical memorandum as necessary once the final rule amending Track Safety Standards is published by the FRA.

Other design guidelines for high-speed facilities are under development and are defined in separate technical memoranda:

- TM 2.3.2: Structure Design Loads [8] defines loading for structures supporting high-speed trains.
- TM 2.1.5: Track Design [9] is the basis for track design.

2.1.2 Design Parameters

- All structures carrying high-speed trains shall be designed to these requirements, and shall comply with the structure gauge and rail section guidelines adopted for the high-speed train system.
- The design life of fixed facilities shall be 100 years per TM 1.1.2 Design Life.
- The maximum initial operating speed is 220 mph. Some segments of the alignment may be designed to not preclude future operation at 250 miles per hour where practical and economically reasonable.
- Structural requirements require that bridges and aerial superstructures be designed as rigid and stiff in order to meet dynamic performance, traffic safety, rail-structure interaction, and passenger comfort requirements.
- Preliminary track design philosophy per TM 2.1.5: Track Design [9] is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.
- Design and construction of high-speed train facilities shall comply with the approved and permitted environmental documents.

- The performance objectives may not be achievable at locations of significant fault rupture. For these cases, variances to the standard design criteria can be made, subject to approval by the Authority, or elevated and underground structures may be prohibited (e.g., tracks shall be at-grade, no exceptions).

2.2 LAWS AND CODES

Initial high-speed rail 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 HSR 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

All bridges and aerial structures that support moving high-speed trains are subject to these requirements to define dynamic performance, provide traffic safety, determine rail-structure interaction, and ensure passenger comfort.

These requirements concern limiting aerial structure and bridge deformations and vibrations which can be magnified under high-speed loading. Excessive deformations and vibrations can lead to numerous issues, including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, ballast instability, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

The basis of the following criteria is informed by EuroCode, specifically EN 1991-2:2003 [2] and EN 1990:2002/A1 [3]. It also used criteria from the Taiwan high-speed rail system [4].

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

Design level analysis requirement are given in Section 3.3.

Table 3-1 summarizes the analysis goals, model type, train loading and speed, desired result, and relevant section.

Table 3-1: Analysis Goals

Analysis Goal	Model Type	Train model	Train speed	Result	Section(s)
Frequency Analysis	Dynamic	--	--	Frequency Evaluation	3.4.2, 3.4.3, 3.4.4
Allowable Vertical Deflection - Static	Static	Single or Multiple Tracks of Modified Cooper E50	--	Static Deflection Limits	3.5.3
Traffic Safety Analysis	Static, LDBE Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation Limits	3.5.6, 3.5.7, 3.5.8
Rail-Structure Interaction Analysis	Static (linear or non-linear), LDBE Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation And Rail Stress Limits	3.6.3, 3.6.4. 3.6.5, 3.6.6
Dynamic Analysis Using Actual High-Speed Trains	Dynamic	Single or Multiple Tracks of Actual High-Speed Trains	90mph to 1.2 Line Speed or 250mph (whichever is less)	Dynamic Impact Factor, Deck Acceleration, Deformation Limits	3.7.4, 3.7.5, 3.7.6, 3.7.7, 3.7.8
Passenger Comfort	Dynamic (Structure & Trainset)	Single Track of Actual High-Speed Trains	90mph to 1.2 Line Speed or 250mph (whichever is less)	Passenger Comfort Limits	3.8.3, 3.8.4, 3.8.5

3.2 DESIGN CODES AND SPECIFICATIONS

There are currently no FRA approved design codes and specifications for high-speed track safety for speeds up to 220 mph. Proposed FRA rules [5] are pending. In absence of final FRA rules, this technical memorandum uses guidance drawn from the following references:

1. European Standard EN 1991-2:2003 Traffic Loads on Bridges [2]
2. European Standard EN 1990:2002/A1 Amendment A1 to EN 1990:2002 [3]
3. Taiwan High Speed Rail (THSR) Corporation: Volume 9, Sections 1, 2, 3, and 9 [4]
4. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009 [1]

3.3 ANALYSIS REQUIREMENTS

3.3.1 General

At part of LRFD force based design, static analysis is required for Cooper E-50 maintenance and construction trains (LLRR), vertical impact effects (I), and load combinations. See TM 2.3.2: Structural Design Loads for details.

This technical memorandum requires additional analyses in order to determine basic structural proportioning, provide good ridability, minimize the probability of derailment, ensure traffic safety, and provide passenger comfort.

Frequency analysis is required for preliminary and final design.

Should a structure fall within the desired vertical frequency range (Section 3.4.2), then:

- 30% design: traffic safety limits per Section 3.5, and rail-structure interaction per Section 3.6 shall apply.
- final design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, and dynamic analysis using actual high-speed trains per Section 3.7 shall apply.

Should a structure fall outside of the desired vertical frequency range (Section 3.4.2), then:

- 30% design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, and dynamic analysis using actual high-speed trains per Section 3.7 shall apply.
- final design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, dynamic analysis using actual high-speed trains per Section 3.7, and passenger comfort analysis per Section 3.9 shall apply.

The final design requirements are subject to change.

3.4 FREQUENCY ANALYSIS

3.4.1 General

Frequency analysis is required in order to place limits on fundamental vibrational characteristics (i.e., mode shapes) of the bridge and aerial structures, in order to ensure well proportioned structures, and minimize resonancy effects.

The desired vertical frequency range shown is known to favorably resist high-speed train actions. It is recommended that structures be proportioned to fall within this range.

Should a structure fall within the desired vertical frequency range, <u>less</u> extensive static and dynamic analysis will be required at preliminary and final design.

Should a structure fall outside of the desired vertical frequency range, <u>more</u> extensive static and dynamic analysis will be required at preliminary and final design..

All frequency analysis shall consider the flexibility of superstructure, bearings, columns, and foundations.

For frequency analysis, two conditions must be investigated:

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

Details of modeling requirements are given in Section 3.9.

3.4.2 Desired Range of Vertical Frequency

The desired range for the first natural frequency of vertical deflection, n_{vert} [Hz], primarily due to bending of the span is [2, Section 6.4.4] of:

$\eta_{\text{lower}} \leq \eta_{\text{vert}} \leq \eta_{\text{upper}}$, where

n_{lower} = 262.5/L for 13 ft ≤ L ≤ 66 ft, or

n_{lower} = 47.645$L^{-0.592}$ for 66 ft ≤ L ≤ 328 ft, and

n_{upper} = 230.46$L^{-0.748}$, for 13 ft ≤ L ≤ 328 ft, and

L = length of simply supported span (ft)

For continuous spans, the effective length, L, shall be:

$L = k * L_{\text{average}}$, where

$L_{\text{average}} = \frac{1}{n}(L_1 + L_2 + ... + L_n)$ = the average span length,

$n =$ the number of spans,

and

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

See Figure 3-1 for the acceptable range.

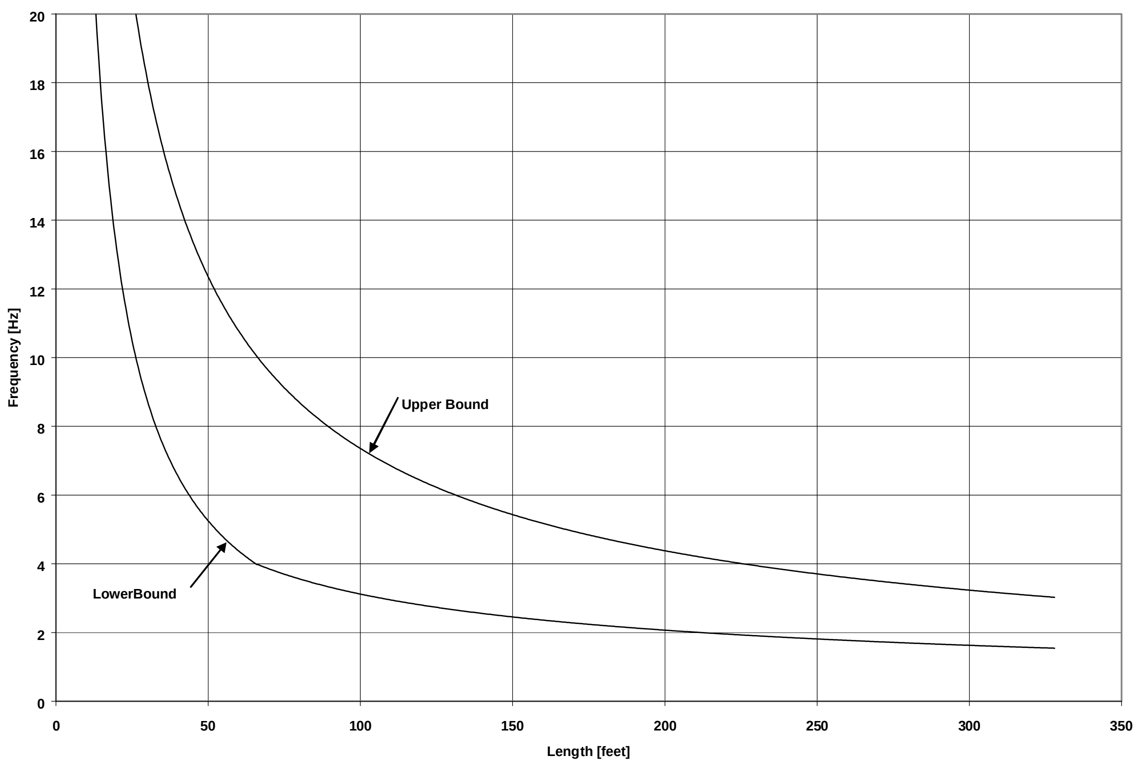


Figure 3-1: Allowable Range of Vertical Frequency

3.4.3 Allowable Transverse Frequency

The first natural frequency of transverse deflection, n_{trans}, of the span shall not be less than 1.2 Hz [3, Section A2.4.4.2.4].

3.4.4 Allowable Torsional Frequency

The first torsional frequency, $n_{torsion}$, of the span shall be greater than 1.2 times the first natural frequency of vertical deflection, n_{vert} [2, Section 6.4.4].

3.5 TRAFFIC SAFETY ANALYSIS

3.5.1 General

Traffic safety analysis, using modified Cooper E-50 loading, provides limits to allowable structural deformation. See Section 3.3 to determine when traffic safety analysis is required at the various levels of design.

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

The flexibility of superstructure, bearings, columns, and foundations shall be considered in traffic safety analysis.

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

Details of modeling requirements are given in Section 3.9.

3.5.2 Modified Cooper E-50 Loading (LLRM)

Modified Cooper E-50 loading (LLRM) shall be used for traffic safety analysis for deformation limits, and in rail-structure interaction analysis for deformation and rail stress limits.

The modified Cooper E-50 loading consists four point loads of 50 kips, along with 5 k/ft uniform load acting elsewhere over a distance of 1000 ft for braking train, and 100 ft for accelerating train, see Figure 3-2.

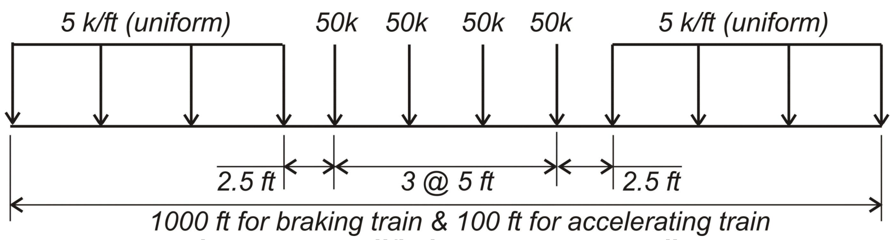


Figure 3-2: Modified Cooper E-50 Loading

3.5.3 Vertical Static Deflection in Span due to LLRM + Impact

<u>For multiple tracks loaded:</u>

For structures with two or more tracks, the maximum static vertical deck deflection (Δ_{LLRM+I}) due to multiple tracks of modified Cooper E-50 loading plus impact (LLRM + I), in their most unfavorable position, shall not exceed L/750, where L = span [3, Section A2.4.4.2.3].

<u>For single track loaded:</u>

For structures with one or more tracks, the maximum static vertical deck deflection (Δ_{LLRM+I}) due to a single track of modified Cooper E-50 loading plus impact (LLRM + I), in the most unfavorable position, shall not exceed [4, Volume 9, Section 3.10.3.1]:

1. For bridges of one to five spans:

$$\Delta_{LLRM+I} \le \frac{L}{2200}$$, where 0 ft < L ≤ 150 ft, or

$$\Delta_{LLRM+I} \le \frac{L}{3000}$$, where 150 ft < L < 390 ft.

2. For bridges of more than five spans, and overall structure length $L_T \le 2625$ ft

$$\Delta_{LLRM+I} \le \frac{L}{2200}\sqrt{\frac{5L+548}{L_T+548}}$$, where L ≤ 390 ft, and L_T = sum of all spans (ft)

3. For bridges of more than five spans, and overall structure length $L_T > 2625$ ft

$$\Delta_{LLRM+I} \le \frac{L}{2200}\sqrt{\frac{5L+548}{3173}}$$, where L ≤ 390 ft, and L_T = sum of all spans (ft)

3.5.4 Traffic Safety Load Cases

Traffic safety loads cases shall include [4, Volume 9, Section 3.6.1.1] :

- Group 1: (LLRM + I) + CF + SF
- Group 2: $(LLRM + I)_1 + CF_1 + SF + WS + WL_1$
- Group 3: $(LLRM + I)_1 + CF_1 + LDBE$

where:

(LLRM + I) = multiple tracks of (LLRM + I) per Section 3.5.5

$(LLRM + I)_1$ = single track of (LLRM + I)

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (single track) per TM 2.3.2: Structure Design Loads

SF = stream flow per TM 2.3.2: Structure Design Loads

WS & WL_1 = wind on structure and one train per TM 2.3.2: Structure Design Loads

LDBE = lower level design earthquake per TM 2.3.2: Structure Design Loads

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

For determining the LDBE demands in Group 3, either equivalent static analysis, dynamic response spectum, or dynamic linear or non-linear time history analysis may be used. See TM 2.10.4: Interim Seismic Criteria for LDBE modeling requirements.

Non-linear track-structure interaction modeling (see Section 3.9.6) is not required, but may be used. For Group 3, superposition of static (i.e., $(LLRM +1)_1 + CF_1$) and either static or dynamic LDBE is allowed.

3.5.5 Multiple Track Loading

For Group 1, where a structure supports multiple tracks, the loading shall be applied for those number of tracks either simultaneously or individually, whichever governs design.

3.5.6 Vertical and Lateral Angular Deformation Limits

Vertical and lateral angular deformation (see Figure 3-3) for Groups 1, 2, and 3 loadings are shown in Table 3-2 [4, Section 3, Appendix B].

Table 3-2: Vertical and Lateral Angular Deformation Limits

Span in ft	Vertical Angle θ/1000 (radians)	Lateral Angle θ/1000 (radians)
≤ 60	1.7	1.7
100	1.5	1.7
≥ 130	1.3	1.3

Intermediate values may be determined by linear interpolation

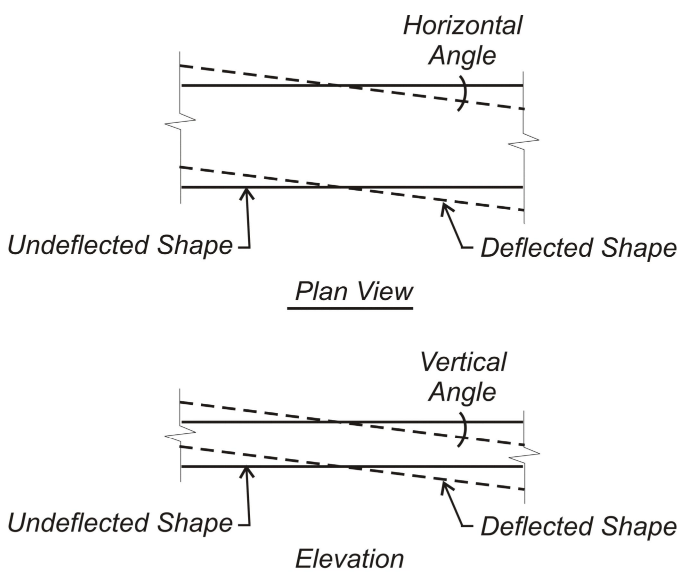


Figure 3-3: Definition of Angular Change

3.5.7 Deck Twist below Tracks

Maximum deck twist below track shall be found for Groups 1, 2 and 3 load cases.

The deck twist, t, is defined as the relative vertical deck displacment under a gauge (s) of 4.75 ft over a length of 10 ft (see Figure 3-4).

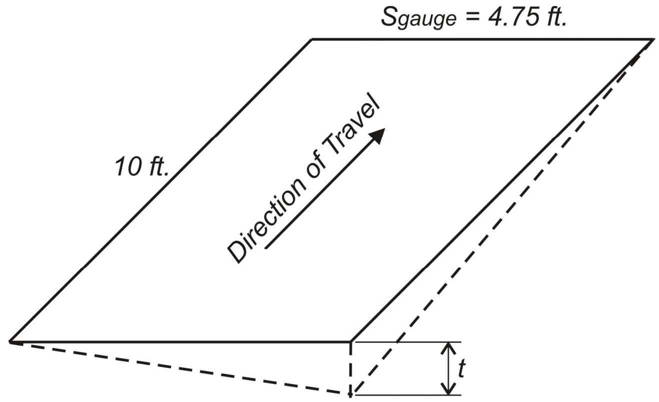


Figure 3-4: Deck Twist Diagram

The maximum allowable deck twist, t_{max} as a function of design line speed is shown in Table 3-3 [3, Section A.2.4.4.2.2].

Table 3-3: Deck Twist Limits (Groups 1, 2, and 3)

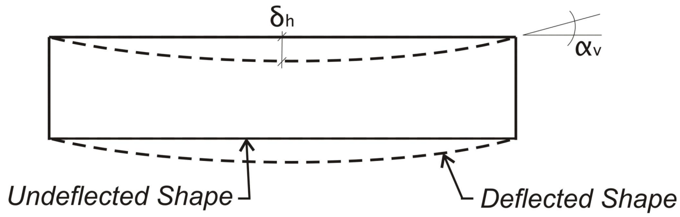
Design Line Speed	t_{max} (in/10ft)
V ≤ 75 mph	0.18
75 < V ≤ 125 mph	0.12
V > 125 mph	0.06

3.5.8 Transverse Deformation Limits

Maximum transverse deformation shall be limited under Groups 1, 2, and 3 loadings [3, Section A2.4.4.2.4, Table A2.8] as follows:

- δ_h, the transverse deflection at top of deck shall be limited so the change in radius of curvature within the deck, $\Delta_r \leq 57{,}420$ ft, where $\Delta_r = L^2/(8\delta_h)$.
- α_v, the end of deck horizontal rotation (i.e., rotation about vertical axis), shall be ≤ 0.0015 radians.

See Figure 3-5 for a schematic of transverse deformation limits.



Plan View

Figure 3-5: Transverse Deformation Limits

3.6 RAIL-STRUCTURE INTERACTION ANALYSIS

3.6.1 General

Rail-structure interaction analysis, using modified Cooper E-50 loading, provides limits to allowable relative longitudinal deformation at expansion joints, and rail stress. Rail-structure interaction analysis is required to minimize the probability of derailment, and ensure good ridability. See Section 3.3 to determine when rail-structure interaction analysis is required at the various levels of design.

The flexibility of superstructure, bearings, columns, and foundations shall be considered in rail-structure interaction analysis.

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

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

Details of rail-structure modeling requirements are given in Section 3.9.6.

3.6.2 Rail-Structure Interaction Load Cases

Rail-structure interaction load cases include [4, Volume 9, Section 3.6.1.1] :

- Group 4: (LLRM + I) + LF
- Group 4r: (LLRM + I) + LF ± TD
- Group 5: $(LLRM + I)_1 + LF_1$ + LDBE
- Group 5r: $(LLRM + I)_1 + LF_1$ ± TD + LDBE

where:

(LLRM + I) = multiple tracks of (LLRM + I) per Section 3.5.5

$(LLRM + I)_1$ = single track of (LLRM + I)

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (single track) per TM 2.3.2: Structure Design Loads

LF = acceleration and braking forces (where two or more tracks, apply braking to one track, and acceleration to one of the other tracks) for LLV loading per TM 2.3.2: Structure Design Loads

LF_1 = acceleration and braking forces (where one track, apply braking) for LLV loading per TM 2.3.2: Structure Design Loads

TD = temperature difference of ±35˚F between rails and deck.

LDBE = lower level design earthquake per TM 2.3.2: Structure Design Loads

Groups 4 and 5 are to provide relative longitudinal deformation limits at expansion joints, and the design of uplift at direct fixation rail.

Groups 4r and 5r include a temperature gradient (± TD), and are used to limit rail stresses.

Static analysis and linear superposition of results, neglecting the non-linear effects of rail-structure interaction, is allowed. Experience has shown this superposition approach to be very conservative, resulting in unrealistically high demands.

Requirements for non-linear track-structure interaction modeling are given in Section 3.9.6.

For determining the LDBE demands in Groups 5 and 5r, either equivalent static analysis, dynamic response spectum, or dynamic linear or non-linear time history analysis may be used. See TM 2.10.4: Interim Seismic Criteria [10] for LDBE modeling requirements. Note that non-linear time-history LDBE analysis (including non-linear rail-structure interaction) may be necessary to substantiate design. In this case, $(LLRM + I)_1 + LF_1$, may be idealized as a set of stationary load vectors placed upon the structure in the most unfavorable position.

3.6.3 Relative Longitudinal Displacement at Expansion Joints

The maximum relative longitudinal displacement between adjacent deck ends (or deck end and abutment), δ_{EXP}, is comprised of separate components:

- δ_{LF} = component due to acceleration and braking alone,
- δ_{LLRM+I} = component due to vertical effects (i.e., end rotations of spans at structural expansion joints), and
- δ_{LDBE} = component due to LDBE alone

For Group 4:

- max δ_{LF} = 0.20” [2, Section 6.5.4.5.2(1)],
- max δ_{LLRM+I} = 0.30” [2, Section 6.5.4.5.2(2)], therefore
- max $\delta_{EXP} \leq$ 0.50”

For Group 5:

- max δ_{LF} = 0.20” [2, Section 6.5.4.5.2(1)],
- max δ_{LLRM+I} = 0.30” [2, Section 6.5.4.5.2(2)],
- max δ_{LDBE} = 0.50”, therefore,
- max $\delta_{EXP} \leq$ 1.0” [4, Volume 3, Section 3.6.1.1]

These limits are for continuous welded rails (ballasted or direct fixation) without rail expansion joints [2, Section 6.5.4.5.2].

The component of relative longitudinal displacement due to vertical effects, $\delta_{LLRM+I,}$ corresponds to the following end rotation limits at expansion joints under either one or two tracks loaded [4, Volume 3, Section 3.9.1].

For ballasted and slab tracks, with long welded rails, the end rotation of the bridge due to vertical effects only, LLRM + I, at the expansion joints is limited to:

$\theta = \left[\dfrac{2.625x10^{-2}}{h(ft)} \right] radians$, for transitions between the decks and abutments

$\theta_1 + \theta_2 = \left[\dfrac{2.625x10^{-2}}{h(ft)} \right] radians$, between consecutive decks with equal superstructure heights.

$\theta_1 + \theta_2 = \left[\dfrac{1.312x10^{-2}}{h_1(ft)} \right] + \left[\dfrac{1.312x10^{-2}}{h_2(ft)} \right] radians$, between consecutive decks with different superstructure heights.

Where,

h(ft): the distance between the top of rail and the center of the bridge bearing

h_1(ft): the distance between the top of rail and the center of the first bridge bearing

h_2(ft): the distance between the top of rail and the center of the second bridge bearing.

See Figure 3-6 for the definition of end rotations at expansion joints.

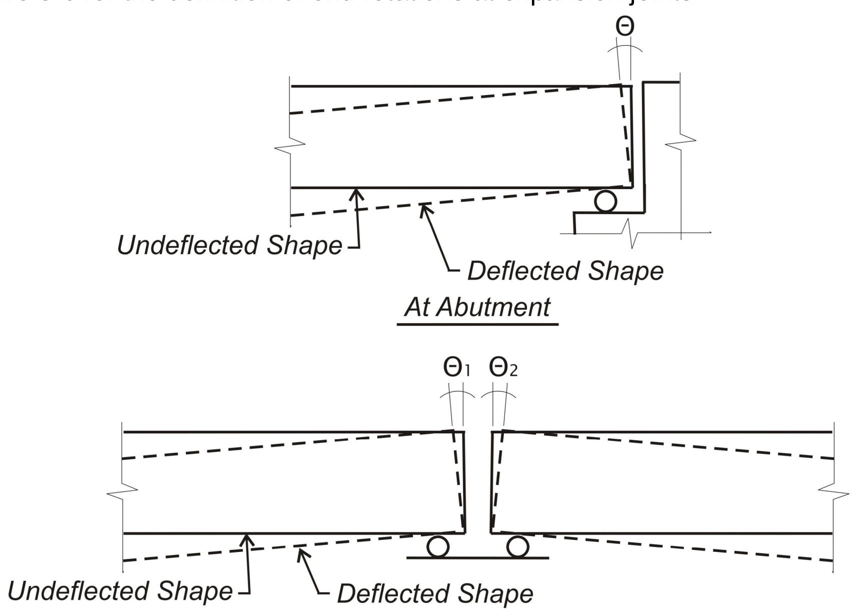


Figure 3-6: End Rotations at Expansion Joints

3.6.4 Relative Vertical Displacement at Bridge Ends

The relative vertical deck deflection, δ_V, between the bridge deck end and adjacent deck or abutment shall be limited for traffic safety.

For Group 4:

- max δ_V = 0.08" [2, Section 6.5.4.5.2(3)],

For Group 5:

- max δ_V = 0.16"

3.6.5 Uplift at Direct Fixation Rail

For Group 5 loading, where direct fixation used, the fastening system capacity shall be designed to withstand any calculated uplift force (F_{uplift}) by a factor safety of 2.

3.6.6 Rail Stress Range Limits

For rails on the bridge or aerial structure and adjacent abutments or at-grade regions, the maximum allowable rail stress range limits, including TD, are [4, Volume 3, Section 3.6.1.1]:

For Group 4f: -10.0 ksi ≤ σ ≤ 13.0 ksi

For Group 5f: -20.0 ksi ≤ σ ≤ 24.0 ksi

Where negative is compression and positive is tension.

Note that the during evaluation of the rail stress range, any stress effects due to pre-heating at the time of construction shall be considered as per TM 2.1.5: Track Design.

These limiting values for rail stress range limits are valid for:

- Standard or high strength rail with a yield strength of at least 74.0 ksi and a tensile strength of 142.5 ksi. For project rail type recommendations, see TM 2.1.5: Track Design.
- The maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.
- Straight track or track radius r ≥ 4920 ft
- Direct fixation slab track with maximum fastener spacing of 24".
- Ballasted track with heavy concrete sleepers of maximum spacing of 24 inches or equivalent track construction and a minimum of 12 inches of consolidated ballast under the sleepers.

For rail fasteners, the non-linear force displacement characteristics in Figure 3-12 or Figure 3-13 shall be the basis for design. Should the fasteners experience plastic deformation ($\Delta_{plastic}$), then sufficient ultimate displacement capacity (Δ_{ult}) shall exist for $\Delta_{ult} > 1.5\Delta_{plastic}$, in order to ensure against fastener fracture.

3.7 Dynamic Analysis using Actual High-Speed Trains

3.7.1 General

Dynamic analysis using actual high-speed trains (LLV) is required in order to detemine resonancy induced dynamic impact (I) effects, limit vertical deck accelerations, deck twist below tracks, end rotations at expansion joints, and relative vertical deflection at bridge ends. See Section 3.3 to determine when dynamic analysis using actual high-speed trains is required at the various levels of design.

All dynamic analysis using actual high-speed trains shall consider the flexibility of superstructure, bearings, columns, and foundations.

Actual representations of high-speed trainsets, running at series of train speeds ranging from 90 mph up to maximum speed of 1.2 times the line design speed shall be considered.

Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loading.

In order to avoid over or underestimating the resonant speeds, two conditions must be investigated:

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

3.7.2 High Speed Train Loading (LLV)

Dynamic analysis shall be undertaken using characterizations of actual high-speed trainsets (LLV). The trainsets shall be idealized as a series of moving point loads at the axle and bogie spacings. Modeling of the train suspension system is not included in this analysis.

TM 6.1: Selected Train Technologies [5, Section 3.1.1] adopted four trainsets as the basis for engineering studies. These four trainsets, presently below, collectively form LLV loading. Each trainset must be investigated individually.

(Note: 17 metric tonnes = 37.48 kips, 14 metric tonnes = 30.87 kips)

AGV (Alstom)

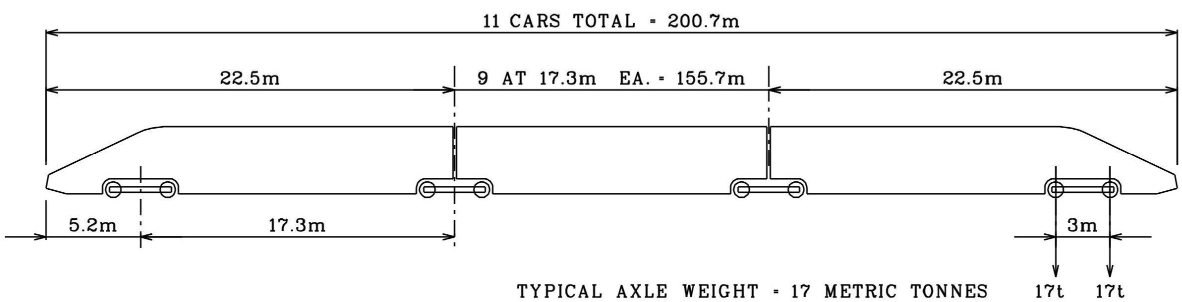


Figure 3-7: AGV (Alston) Loading Diagram

Zefiro (Bombardier)

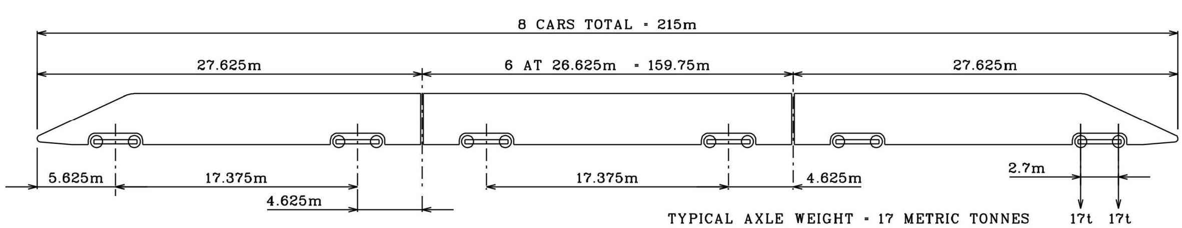


Figure 3-8: Zefiro (Bombardier) Loading Diagram

ICE-3 Velaro (Siemens)

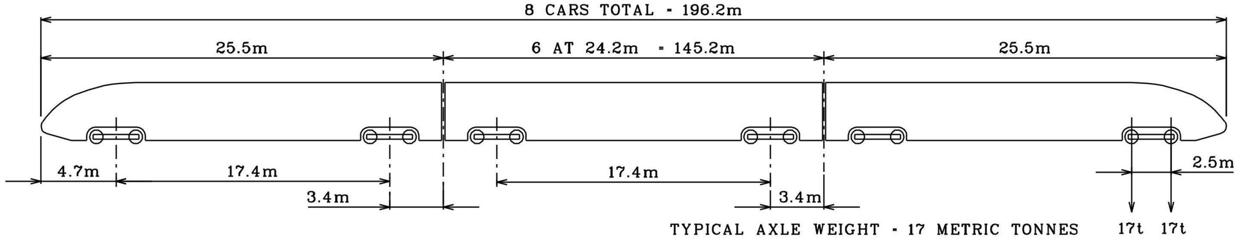


Figure 3-9: ICE-3 Velaro (Siemens) Loading Diagram

N700 (Hitachi)

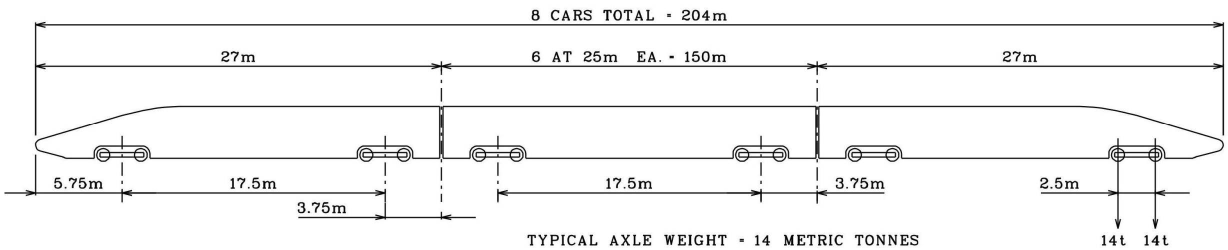


Figure 3-10: N700 (Shinkansen) Loading Diagram

3.7.3 Train Speeds

For each actual high-speed trainset (LLV), a series of speeds ranging from a minimum of 90 mph up to maximum speed of 1.2 times the line design speed [2, Section 6.4.6.2], by increment of 20 mph, shall be run across the dynamic model. Smaller increments of 5 mph shall be used for ±20 mph on each side of resonant speeds, if applicable.

For simple spans, the resonant speeds may be estimated by:

$V_i = n_o d/i$, where V_i = resonant speeds,

n_o = first natural frequency of vertical deflection

d = axle group spacing (i.e., coach spacing)

i = 1, 2, 3, 4, …

For more complicated span arrangements, the resonant speeds shall be determined by the dynamic model.

3.7.4 Dynamic Impact Factors

For the high-speed trainsets (LLV), the dynamic model shall be used to determine the impact factors (I) [2, Section 6.4.6.5].

In order to determine (I), the maximum dynamic response value, ξ_{dyn}, shall be found for each structural response taking into account single track loading (LLV) and multiple speeds.

Comparing with the corresponding static response value, ξ_{stat}, the dynamic impact factor is:

$$I = \max\left[\frac{\xi_{dyn}}{\xi_{stat}}\right] - 1 > 1$$

3.7.5 Vertical Deck Acceleration

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

To determine maximum vertical deck acceleration, an upper bound estimate of stiffness and lower bound estimate of mass shall be used.

Vertical acceleration of bridge and aerial structure decks shall be found for a single track (LLV+I) loaded only, over the range of train speeds given in Section 3.7.3.

The peak values of vertical deck acceleration [3, Section A2.4.4.2.1] are:

- For ballasted deck: 11.3 ft/s^2 (0.35g)
- For direct fixation track: 16.1 ft/s^2 (0.50g)

Note that this pertains to accelerations at the top of deck, for acceleration limits to be experienced by passengers, see Section 3.8.3 below.

3.7.6 Deck Twist below Tracks

The maximum deck twist below track (variation of cant) shall be checked in order to ensure that the four wheel contact points of a bogie are not too far from a plane.

To determine maximum deck twist below track, a lower bound estimate of stiffness and upper bound estimate of mass shall be used.

Maximum deck twist below track shall be found for a single track (LLV+I) loaded only over the range of train speeds given in Section 3.7.3.

The deck twist, t, is defined as the relative vertical deck displacment under a gauge (s) of 4.75 ft over a length of 10 ft (see Figure 3-4).

The maximum allowable deck twist, t_{max} = 0.05 in/10ft, for a single track (LLV+I) only [4, Section 3.9.1].

3.7.7 Relative Longitudinal Displacement at Expansion Joints

Maximum relative longitudinal displacements at expansion joints at expansion joints shall be found for a single or double track of LLV+I, where I is the dynamic impact factor implicit to the dynamic model. This calculation shall be made over the range of train speeds given in Section 3.7.3.

The maximum relative longitudinal displacement between adjacent deck ends (or deck end and abutment), δ_{LLV+I}, is:

δ_{LLV+I} = 0.30" [2, Section 6.5.4.5.2(2)]

This limit is for continuous welded rails (ballasted or direct fixation) without rail expansion joints [2, Section 6.5.4.5.2].

To determine maximum relative longitudinal displacement at expansion joints, a lower bound estimate of stiffness and upper bound estimate of mass shall be used.

This relative longitudinal displacement corresponds to the following end rotation limits at expansion joints [4, Volume 3, Section 3.9.1].

For ballasted and slab tracks, with long welded rails, the end rotation of the bridge at the expansion joints is limited to:

$\theta = \left[\dfrac{2.625x10^{-2}}{h(ft)}\right] radians$, for transitions between the decks and abutments

$\theta_1 + \theta_2 = \left[\dfrac{2.625x10^{-2}}{h(ft)} \right] radians$, between consecutive decks with equal superstructure heights.

$\theta_1 + \theta_2 = \left[\dfrac{1.312x10^{-2}}{h_1(ft)} \right] + \left[\dfrac{1.312x10^{-2}}{h_2(ft)} \right] radians$, between consecutive decks with different superstructure heights.

Where,

h(ft): the distance between the top of rail and the center of the bridge bearing

h_1(ft): the distance between the top of rail and the center of the first bridge bearing

h_2(ft): the distance between the top of rail and the center of the second bridge bearing.

See Figure 3-6 for the definition of end rotations at expansion joints.

3.7.8 Relative Vertical Deflection at Bridge Ends

The relative vertical deck deflection between the bridge deck end and adjacent deck or abutment shall be limited for traffic safety.

To determine the maximum relative vertical deck deflection, a lower bound estimate of stiffness and upper bound estimate of mass shall be used.

Maximum relative vertical deck deflection shall be found for a single or double track (LLV+I) loaded only, over the range of train speeds given in Section 3.7.3.

The maximum relative vertical deck deflection between bridge deck end and adjacent deck or abutment, due to (LLV + I) alone, is 0.08” [2, Section 6.5.4.5.2.3].

3.8 PASSENGER COMFORT ANALYSIS

3.8.1 General

Passenger comfort depends upon both the vertical accelerations and change in vertical accelerations experienced by passengers inside the coach during travel to, on, and off bridges or aerial structures. See Section 3.3 to determine when passenger comfort analysis is required at the various levels of design.

Generally, for typical structures, limiting the maximum vertical span deflection and deck acceleration provides the sufficent guidance for passenger comfort.

Passenger comfort is a final design issue. Should a structure fall outside of the desired vertical frequency range (Section 3.4.2), then passenger comfort analysis will be required at final design.

For exceptional structures (e.g., long span structures, continuous beams with widely varying span lengths, and spans with wide variation in stiffness), passenger comfort analysis may be required, as determined by the Authority.

3.8.2 Dynamic Train-Structure Interaction Analysis

For passenger comfort analysis, both a dynamic model of the structure and a dynamic models of the trainsets shall be used. The coupled interaction of the structure and trainset models shall be considered in either an exact or iterative method.

Details of structural modeling requirements are given in Section 3.9.

The dynamic model of the trainsets (details pending) shall consider the mass, stiffness, and damping characteristics of the trainsets, including wheels, bogies, suspension, and body.

Similar to Section 3.7.3 above, passenger comfort analysis shall consider a series of speeds ranging from a minimum of 90 mph up to maximum speed of 1.2 times the line design speed.

Passenger comfort analysis shall consider single track (i.e., one trainset) loading for all structures.

3.8.3 Maximum Lateral Acceleration Experience by Passengers

The maximum lateral acceleration experienced by passengers within an operating train is limited to 1.6 ft/s^2 (0.05 g).

3.8.4 Maximum Vertical Acceleration Experience by Passengers

The maximum vertical acceleration experienced by passengers within an operating train is limited to 3.2 ft/s^2 (0.10 g).

3.8.5 Maximum Vertical Jerk Experience by Passengers

The first time derivative of vertical acceleration (i.e., rate of change of acceleration) is known as jerk. The maximum vertical jerk experienced by passengers within an operating train is limited to 6.5 ft/s^3 [7].

3.9 MODELING REQUIREMENTS

3.9.1 General

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

3.9.2 Model Geometry and Boundary Conditions

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

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

For repetitive aerial structure viaducts with simply supported spans the model shall have a minimum of twenty (20) spans, with the subject span as the middle span. Boundary conditions at the ends of the model shall represent the stiffness of any adjacent spans or frames.

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

Soil springs at the foundations shall be per the Project Geotechnical Report.

3.9.3 Model Stiffness

Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loads at resonant speed.

In order to avoid overestimating the critical resonant speed, a lower bound estimate of structure stiffness shall be included in the model.

For bridge or aerial structure piers and columns, an upper bound estimate of bending inertia of I_g, and a lower bound estimate of bending inertia of $0.8I_g$ shall be made, where I_g is the gross cross sectional bending inertia.

All structural elements shall be represented by the appropriate sectional properties and constitutive relations. Cracked bending inertias $< 0.8I_g$ for the piers and columns shall be considered, especially for LDBE response, and integral superstructure to pier connections considering cracking due to long term loadings (i.e., creep and shrinkage effects).

3.9.4 Model Mass

Both upper and lower bound estimate of bridge mass shall be considered.

In order to avoid overestimating the critical resonant speed, an upper bound estimate of mass (along with a lower bound estimate of stiffness) shall be made.

In order to avoid underestimating peak deck accelerations, a lower bound estimate of mass (along with an upper bound estimate of stiffness) shall be made.

For structural dead load (DC) mass, use the units weights in TM 2.3.2: Structure Design Loads.

For superimposed dead load (DW), use both upper and lower bound estimates.

For ballasted track, use a lower bound estimate of ballast weight of 100 pcf at minimum ballast thickness, and an upper bound estimate of 150 pcf allowing for future ballast lifts if applicable.

3.9.5 Model Damping

The peak structural response at resonant speed is highly dependent upon damping. Table 3-4 gives the lower bound estimates of damping to be used [2, Section 6.4.6.3.1].

Table 3-4: Damping Values for Dynamic Model

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

These lower bound values may be increased for shorter spans (< 65 ft), see [2].

3.9.6 Modeling of Rail-Structure Interaction

Longitudinal actions produce longitudinal forces in the continuous rails. These forces are distributed to the bridge and aerial structures in accordance with the relative stiffness of the ties and fasteners, articulation of the structural system, and stiffness of the substructure, see Figure 3-11.

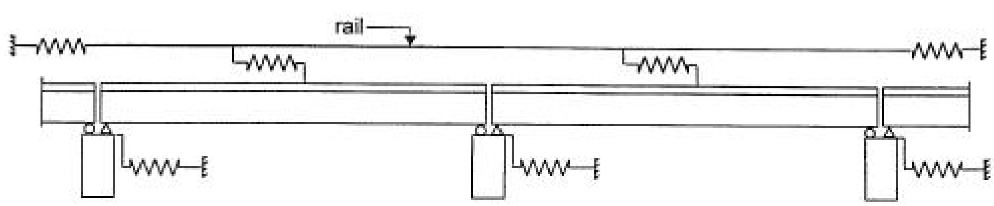


Figure 3-11: Rail-Structure Interaction Model

Rail-structure interaction is important in that it may govern the:

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

Rail-structure interaction shall be performed for all structures using either static or dynamic models. In addition, the model shall include the axial stiffness of the rails appropriately located upon the superstructure, and longitudinal bi-linear coupling springs between the track and superstructure over the length of the model. The bi-linear coupling springs shall represent either the ballast track (see Figure 3-12) or direct fixation slab track (see Figure 3-13) between the rails and superstructure [2, Section 6.5.4.4].

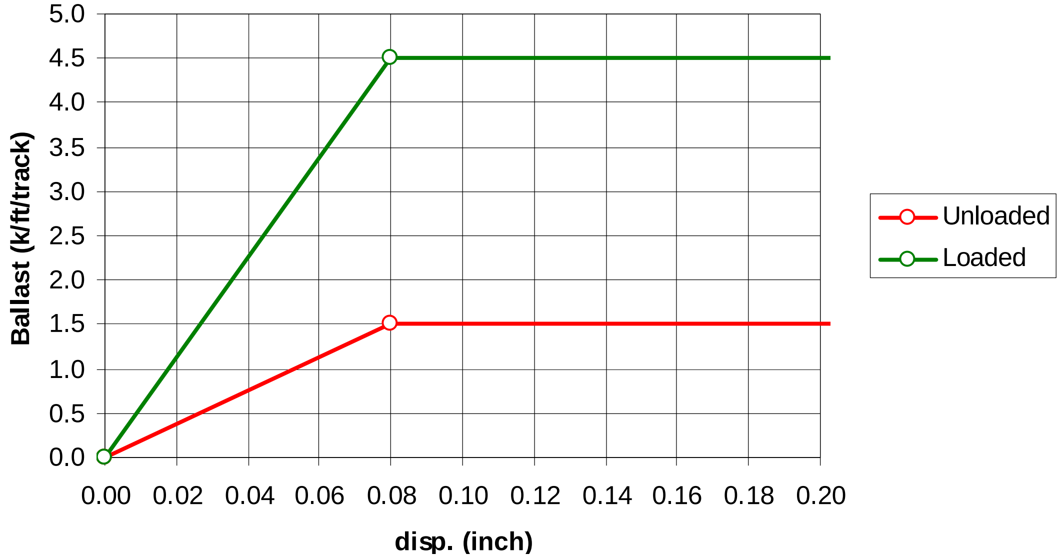


Figure 3-12: Ballasted Track Bi-linear Coupling Springs

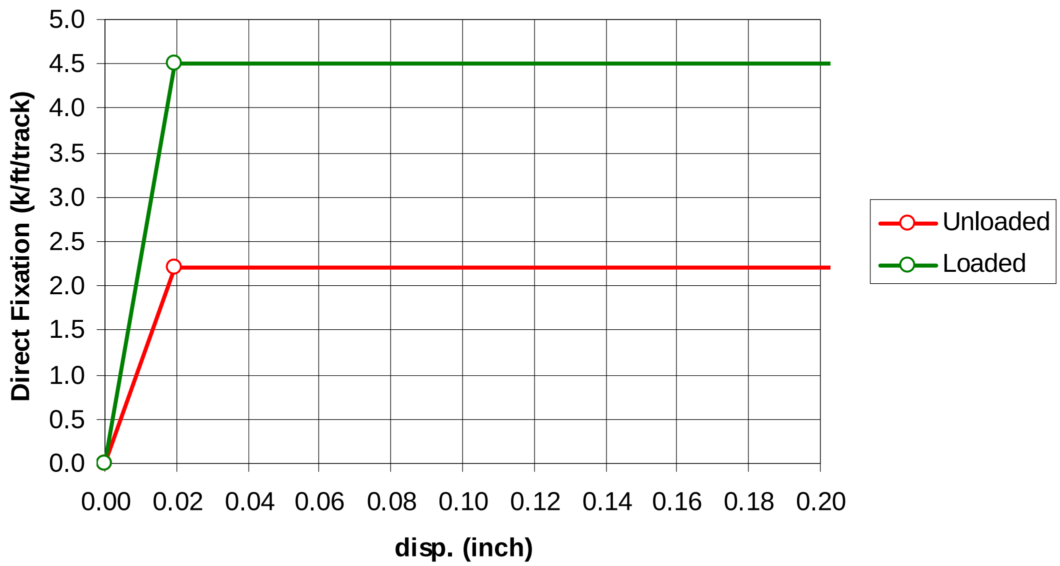


Figure 3-13: Direct Fixation Slab Track Bi-linear Coupling Springs

The total number of springs per each span shall not be less than ten (10) and the spacing between the springs shall not be more than 15 ft.

Where an abutment occurs at the viaduct ends, the rails and coupling springs shall be extended a minimum distance of L+130 ft from the face of the abutment; where L is equal to the average span length.

For a long viaduct the model shall consist of at least 20 spans for both normal operation analysis and earthquake analysis. A horizontal spring with a spring constant of 10,500 kips/ft shall be used at the "dead" end of each track. The yield load of the track end springs shall be equal to q_y*L/4 of the last adjacent span in the model, where q_y is the yield load (k/ft of track) of the rail. For example, q_y equals 1.5 k/ft of track for an unloaded ballast track [4].

3.10 OTHER PENDING ISSUES

3.10.1 General

Other issues will be addressed as the CHST criteria is developed. These other issues include guidelines for repetitive span arrangements, and guidelines for rail break. It is expected that other issues may arise in the future.

3.10.2 Guidelines for Repetitive Span Arrangements

Guidelines are pending.

3.10.3 Guidelines for Rail Break

Guidelines are pending.

3.10.4 Guidelines for Camber due to Creep/Shrinkage

Guidelines are pending.

3.10.5 Guidelines for Settlement at Supports

Guidelines are pending

3.10.6 Guidelines for Analysis of Derailment Loads

Guidelines are pending

3.10.7 Guidelines for Application of Nosing and Hunting Forces

Guidelines are pending

4.0 SUMMARY AND RECOMMENDATIONS

Specific requirements for high-speed track and structure interaction for aerial structures and bridges have been made. These requirements encompass tracks supported by either ballast or direct fixation fasteners. The requirements include dynamic performance, traffic safety, rail-structure interaction, and passenger comfort.

The requirements concern limiting bridge deformations and vibrations, which can be magnified under high-speed loading and lead to numerous issues including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, ballast instability, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

Currently, there are no current Federal Railway Administration (FRA) rules for high-speed (220 mph+) track-structure interaction design. For this reason, this Technical Memorandum relies upon European and Taiwan high-speed rail criteria. The Federal Railway Administration (FRA) has recently published proposed rules [5] addressing aspects of high-speed track-structure interaction for Class 9 (≤ 220 mph) track. The CHSTP will review and revise the guidance in this technical memorandum as necessary once the final rule amending Track Safety Standards is published by the FRA.

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

Frequency analysis is required for preliminary and final design.

Should a structure fall within the desired vertical frequency range (Section 3.4.2), then:

- preliminary design: traffic safety limits per Section 3.5, and rail-structure interaction per Section 3.6 shall apply.
- final design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, and dynamic analysis using actual high-speed trains per Section 3.7 shall apply.

Should a structure fall outside of the desired vertical frequency range (Section 3.4.2), then:

- preliminary design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, and dynamic analysis using actual high-speed trains per Section 3.7 shall apply.
- final design: traffic safety limits per Section 3.5, rail-structure interaction per Section 3.6, dynamic analysis using actual high-speed trains per Section 3.7, and passenger comfort analysis per Section 3.9 shall apply.

These requirements are necessary on a system-wide basis to ensure that performance requirements of high-speed train structures are met, provide a consistent basis for advancing design, and provide designers a common basis in proportioning materials to allow uniform description of construction techniques and cost estimates.

5.0 SOURCE INFORMATION AND REFERENCES

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. European Standard EN 1991-2:2003 Traffic Loads on Bridges – Part 2: Traffic Loads on Bridges.
3. European Standard EN 1990:2002/A1 Amendment A1 to EN 1990:2002
4. Taiwan High-Speed Railway Design Manual (2000), Volume 9, Sections 1, 2, 3, and 9
5. Federal Register Vol. 75, No. 89, to revise Title 49 – Transportation, of the Code of Federal Regulations (CFR); Part 213 - Track Safety Standards, and Part 238 – Passenger Equipment Safety Standards.
6. TM 6.1: Selected Train Technologies
7. ISO 2041
8. CHSTP TM 2.3.2: Structural Design Loads, R1
9. CHSTP TM 2.1.5: Draft Track Design, R0
10. CHSTP TM 2.10.4: Interim Seismic Criteria, R0

6.0 DESIGN MANUAL CRITERIA

6.1 GENERAL

All bridges and aerial structures which support moving high-speed trains are subject to these requirements to define dynamic performance, provide traffic safety, determine rail-structure interaction, and ensure passenger comfort.

These requirements concern limiting aerial structure and bridge deformations and vibrations that can be magnified under high-speed loading. Excessive deformations and vibrations can lead to numerous issues, including unacceptable changes in vertical and horizontal track geometry, excessive rail stress, ballast instability, reduction in wheel contact, dynamic amplification of loads, and passenger discomfort.

The basis of the following criteria is informed by EuroCode, specifically EN 1991-2:2003 [2] and EN 1990:2002/A1 [3]. It also used criteria from the Taiwan high-speed rail system [4].

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

Design level analysis requirement are given in Section 6.3.

Table 6-1 summarizes the analysis goals, model type, train loading and speed, desired result, relevant section.

Table 6-1: Analysis Goals

Analysis Goal	Model Type	Train model	Train speed	Result	Section(s)
Frequency Analysis	Dynamic	--	--	Frequency Evaluation	6.4.2, 6.4.3, 6.4.4
Allowable Vertical Deflection - Static	Static	Single or Multiple Tracks of Modified Cooper E50	--	Static Deflection Limits	6.5.3
Traffic Safety Analysis	Static, LDBE Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation Limits	6.5.6, 6.5.7, 6.5.8
Rail-Structure Interaction Analysis	Static (linear or non-linear), LDBE Static or Dynamic	Single or Multiple Tracks of Modified Cooper E50	--	Deformation And Rail Stress Limits	6.6.3, 6.6.4. 6.6.5, 6.6.6
Dynamic Analysis Using Actual High-Speed Trains	Dynamic	Single or Multiple Tracks of Actual High-Speed Trains	90mph to 1.2 Line Speed or 250mph (whichever is less)	Dynamic Impact Factor, Deck Acceleration, Deformation Limits	Pending
Passenger Comfort	Dynamic (Structure & Trainset)	Single Track of Actual High-Speed Trains	90mph to 1.2 Line Speed or 250mph (whichever is less)	Passenger Comfort Limits	Pending

6.2 DESIGN CODES AND SPECIFICATIONS

There are no currently FRA approved design codes and specifications for high-speed track safety for speeds up to 220 mph. Proposed FRA rules [5] are pending.

In the absence of final FRA rules, this technical memorandum uses guidance drawn from the following references:

1. European Standard EN 1991-2:2003 Traffic Loads on Bridges [2]
2. European Standard EN 1990:2002/A1 Amendment A1 to EN 1990:2002 [3]
3. Taiwan High Speed Rail (THSR) Corporation: Volume 9, Sections 1, 2, 3, and 9 [4]

4. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009 [1]

The CHSTP will review and revise this guidance in this technical memorandum as necessary once the final rule amending Track Safety Standards is published by the FRA.

6.3 ANALYSIS REQUIREMENTS

6.3.1 General

At part of LRFD force based design, static analysis is required for Cooper E-50 maintenance and construction trains (LLRR), vertical impact effects (I), and load combinations. See TM 2.3.2: Structural Design Loads for details..

This technical memorandum requires additional analyses in order to determine basic structural proportioning, provide good ridability, minimize the probability of derailment, ensure traffic safety, and provide passenger comfort.

Frequency analysis is required for preliminary and final design.

All structures shall fall within the desired vertical frequency range (Section 6.4.2), and comply with the following requirements:

- Preliminary design: traffic safety limits per Section 6.5, and rail-structure interaction per Section 6.6.
- Final design: traffic safety limits per Section 6.5, rail-structure interaction per Section 6.6, and dynamic analysis using actual high-speed trains. Requirements for dynamic analysis will be defined in future revisions of this document.

The final design requirements are subject to change.

6.4 FREQUENCY ANALYSIS

6.4.1 General

Frequency analysis is required in order to place limits on fundamental vibrational characteristics (i.e., mode shapes) of the bridge and aerial structures, in order to ensure well proportioned structures, and minimize resonancy effects.

The desired vertical frequency range shown is known to favorably resist high-speed train actions. It is required that structures be proportioned to fall within this range.

All frequency analysis shall consider the flexibility of superstructure, bearings, columns, and foundations.

For frequency analysis, two conditions must be investigated:

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

Details of modeling requirements are given in Section 6.9.

6.4.2 Desired Range of Vertical Frequency

The desired range for the first natural frequency of vertical deflection, n_{vert} [Hz], primarily due to bending of the span is [2, Section 6.4.4]:

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

$n_{lower} = 262.5/L$ for 13 ft ≤ L ≤ 66 ft, or

$n_{lower} = 47.645L^{-0.592}$ for 66 ft ≤ L ≤ 328 ft, and

$n_{upper} = 230.46L^{-0.748}$, for 13 ft ≤ L ≤ 328 ft, and

L = length of simply supported span (ft)

For continuous spans, the effective length, L, shall be:

$L = k * L_{\text{average}}$, where

$L_{\text{average}} = \frac{1}{n}(L_1 + L_2 + ... + L_n)$ = the average span length,

$n =$ the number of spans,

and

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

See Figure 3-1 for the acceptable range.

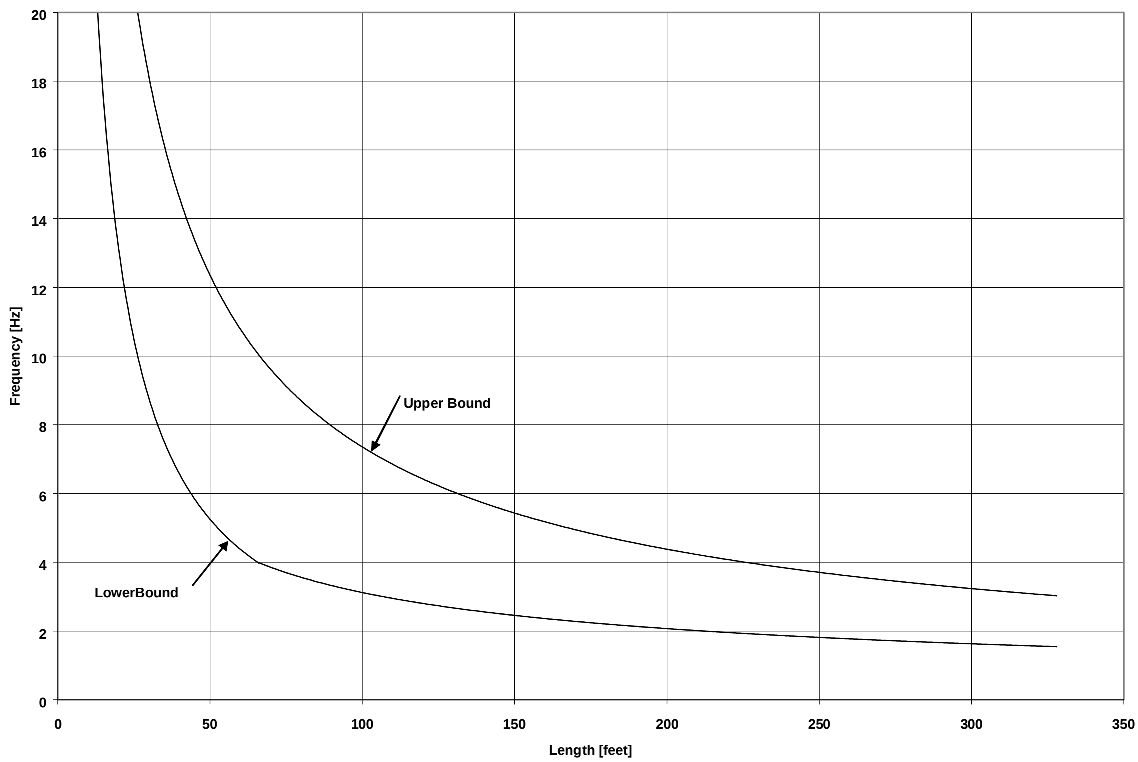


Figure 6-1: Allowable Range of Vertical Frequency

6.4.3 Allowable Transverse Frequency

The first natural frequency of transverse deflection, n_{trans}, of the span shall not be less than 1.2 Hz [3, Section A2.4.4.2.4].

6.4.4 Allowable Torsional Frequency

The first torsional frequency, n_{torsion}, of the span shall be greater than 1.2 times the first natural frequency of vertical deflection, n_{vert} [2, Section 6.4.4].

6.5 TRAFFIC SAFETY ANALYSIS

6.5.1 General

Traffic safety analysis, using modified Cooper E-50 loading, provides limits to allowable structural deformation. See Section 6.3 to determine when traffic safety analysis is required at the various levels of design.

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

The flexibility of superstructure, bearings, columns, and foundations shall be considered in traffic safety analysis.

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

Details of modeling requirements are given in Section 6.9.

6.5.2 Modified Cooper E-50 Loading (LLRM)

Modified Cooper E-50 loading (LLRM) shall be used for traffic safety analysis for deformation limits, and in rail-structure interaction analysis for deformation and rail stress limits.

The modified Cooper E-50 loading consists four point loads of 50 kips, along with 5 k/ft uniform load acting elsewhere over a distance of 1000 ft for braking train, and 100 ft for accelerating train, see Figure 3-2.

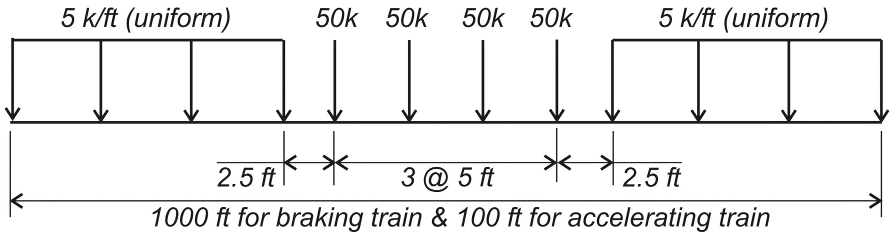


Figure 6-2: Modified Cooper E-50 Loading

6.5.3 Vertical Static Deflection in Span due to LLRM + Impact

<u>For multiple tracks loaded:</u>

For structures with two or more tracks, the maximum static vertical deck deflection (Δ_{LLRM+I}) due to multiple tracks of modified Cooper E-50 loading plus impact (LLRM + I), in their most unfavorable position, shall not exceed L/750, where L = span [3, Section A2.4.4.2.3].

<u>For single track loaded:</u>

For structures with one or more tracks, the maximum static vertical deck deflection (Δ_{LLRM+I}) due to a single track of modified Cooper E-50 loading plus impact (LLRM + I), in the most unfavorable position, shall not exceed [4, Volume 9, Section 3.10.3.1]:

1. For bridges of one to five spans:

$$\Delta_{LLRM+I} \leq \frac{L}{2200}$$, where 0 ft < L ≤ 150 ft, or

$$\Delta_{LLRM+I} \leq \frac{L}{3000}$$, where 150 ft < L < 390 ft.

2. For bridges of more than five spans, and overall structure length $L_T \leq 2625$ ft

$$\Delta_{LLRM+I} \leq \frac{L}{2200}\sqrt{\frac{5L+548}{L_T+548}}$$, where L ≤ 390 ft, and L_T = sum of all spans (ft)

3. For bridges of more than five spans, and overall structure length $L_T > 2625$ ft

$$\Delta_{LLRM+I} \leq \frac{L}{2200}\sqrt{\frac{5L+548}{3173}}$$, where L ≤ 390 ft, and L_T = sum of all spans (ft)

.

6.5.4 Traffic Safety Load Cases

Traffic safety loads cases shall include [4, Volume 9, Section 3.6.1.1] :

- Group 1: (LLRM + I) + CF + SF
- Group 2: $(LLRM + I)_1 + CF_1 + SF + WS + WL_1$
- Group 3: $(LLRM + I)_1 + CF_1 + LDBE$

where:

(LLRM + I) = multiple tracks of (LLRM + I) per Section 6.5.5

$(LLRM + I)_1$ = single track of (LLRM + I)

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (single track) per TM 2.3.2: Structure Design Loads

SF = stream flow per TM 2.3.2: Structure Design Loads

WS & WL_1 = wind on structure and one train per TM 2.3.2: Structure Design Loads

LDBE = lower level design earthquake per TM 2.3.2: Structure Design Loads

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

For determining the LDBE demands in Group 3, either equivalent static analysis, dynamic response spectum, or dynamic linear or non-linear time history analysis may be used. See TM 2.10.4: Interim Seismic Criteria for LDBE modeling requirements.

Non-linear track-structure interaction modeling (see Section 6.9.6) is not required, but may be used. For Group 3, superposition of static (i.e., $(LLRM +1)_1 + CF_1$) and either static or dynamic LDBE is allowed.

6.5.5 Multiple Track Loading

For Group 1, where a structure supports multiple tracks, the loading shall be applied for those number of tracks either simultaneously or individually, whichever governs design

6.5.6 Vertical and Lateral Angular Deformation Limits

Vertical and lateral angular deformation (see Figure 6-3) for Groups 1, 2, and 3 loadings are shown in Table 6-2 [4, Section 3, Appendix B].

Table 6-2: Vertical and Lateral Angular Deformation Limits

Span in ft	Vertical Angle θ/1000 (radians)	Lateral Angle θ/1000 (radians)
≤ 60	1.7	1.7
100	1.5	1.7
≥ 130	1.3	1.3

Intermediate values may be determined by linear interpolation

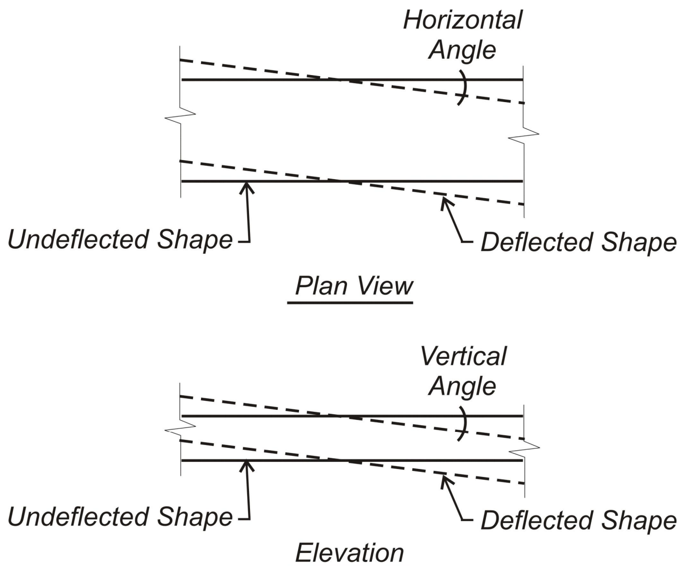


Figure 6-3: Definition of Angular Change

6.5.7 Deck Twist below Tracks

Maximum deck twist below track shall be found for Groups 1, 2 and 3 load cases.

The deck twist, t, is defined as the relative vertical deck displacment under a gauge (s) of 4.75 ft over a length of 10 ft (see Figure 6-4).

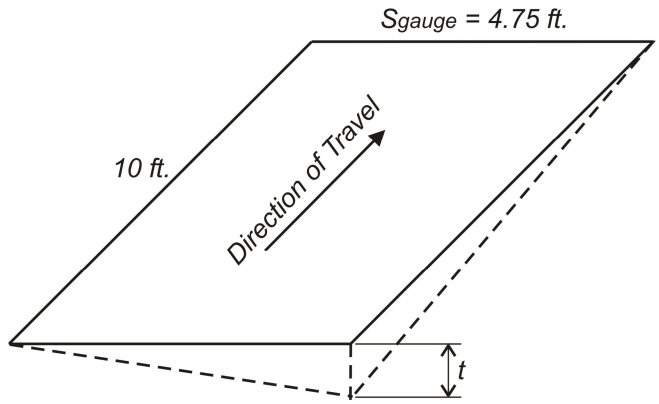


Figure 6-4: Deck Twist Diagram

The maximum allowable deck twist, t_{max} as a function of design line speed is shown in Table 6-3 [3, Section A.2.4.4.2.2].

Table 6-3: Deck Twist Limits (Groups 1, 2, and 3)

Design Line Speed	t_{max} (in/10ft)
V ≤ 75 mph	0.18
75 < V ≤ 125 mph	0.12
V > 125 mph	0.06

6.5.8 Transverse Deformation Limits

Maximum transverse deformation shall be limited under Groups 1, 2, and 3 loadings [3, Section A2.4.4.2.4, Table A2.8] as follows:

- δ_h, the transverse deflection at top of deck shall be limited so the change in radius of curvature within the deck, $\Delta_r \leq 57{,}420$ ft, where $\Delta_r = L^2/(8\delta_h)$.
- α_v, the end of deck horizontal rotation (i.e., rotation about vertical axis), shall be ≤ 0.0015 radians.

See Figure 6-5 for a schematic of transverse deformation limits.

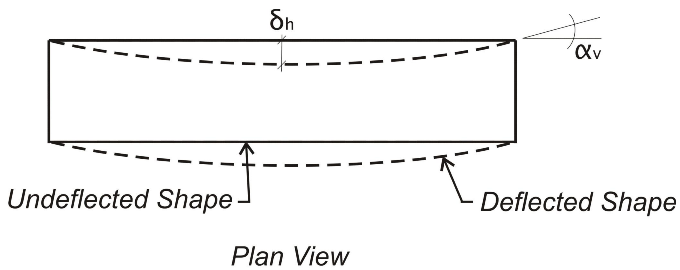


Figure 6-5: Transverse Deformation Limits

6.6 RAIL-STRUCTURE INTERACTION ANALYSIS

6.6.1 General

Rail-structure interaction analysis, using modified Cooper E-50 loading, provides limits to allowable relative longitudinal deformation at expansion joints, and rail stress. Rail-structure interaction analysis is required to minimize the probability of derailment, and ensure good ridability. See Section 6.3 to determine when rail-structure interaction analysis is required at the various levels of design.

The flexibility of superstructure, bearings, columns, and foundations shall be considered in rail-structure interaction analysis.

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

Preliminary track design philosophy per TM 2.1.5: Track Design, is to avoid rail expansion joints if practical. Thus, for preliminary design, the maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.

Details of rail-structure modeling requirements are given in Section 6.9.6.

6.6.2 Rail-Structure Interaction Load Cases

Rail-structure interaction load cases include [4, Volume 9, Section 3.6.1.1] :

- Group 4: (LLRM + I) + LF
- Group 4r: (LLRM + I) + LF ± TD
- Group 5: $(LLRM + I)_1 + LF_1$ + LDBE
- Group 5r: $(LLRM + I)_1 + LF_1$ ± TD + LDBE

where:

(LLRM + I) = multiple tracks of (LLRM + I) per Section 6.5.5

$(LLRM + I)_1$ = single track of (LLRM + I)

I = vertical impact factor from LLRR per TM 2.3.2: Structure Design Loads

CF = centrifugal force (multiple tracks) per TM 2.3.2: Structure Design Loads

CF_1 = centrifugal force (single track) per TM 2.3.2: Structure Design Loads

LF = acceleration and braking forces (where two or more tracks, apply braking to one track, and acceleration to one of the other tracks) for LLV loading per TM 2.3.2: Structure Design Loads

LF_1 = acceleration and braking forces (where one track, apply braking) for LLV loading per TM 2.3.2: Structure Design Loads

TD = temperature difference in ±35˚F between rails and deck.

LDBE = lower level design earthquake per TM 2.3.2: Structure Design Loads

Groups 4 and 5 are to provide relative longitudinal deformation limits at expansion joints, and the design of uplift at direct fixation rail.

Groups 4r and 5r include a temperature gradient (± TD), and are used to limit rail stresses.

Static analysis and linear superposition of results, neglecting the non-linear effects of rail-structure interaction, is allowed. Experience has shown this superposition approach to be very conservative, resulting in unrealistically high demands.

Requirements for non-linear track-structure interaction modeling are given in Section 6.9.6.

For determining the LDBE demands in Groups 5 and 5r, either equivalent static analysis, dynamic response spectum, or dynamic linear or non-linear time history analysis may be used. See TM 2.10.4: Interim Seismic Criteria [10] for LDBE modeling requirements. Note that non-linear time-history LDBE analysis (including non-linear rail-structure interaction) may be necessary to substantiate design. In this case, $(LLRM + I)_1 + LF_1$, may be idealized as a set of stationary load vectors placed upon the structure in the most unfavorable position.

6.6.3 Relative Longitudinal Displacement at Expansion Joints

The maximum relative longitudinal displacement between adjacent deck ends (or deck end and abutment), δ_{EXP}, is comprised of separate components:

- δ_{LF} = component due to acceleration and braking alone,
- δ_{LLRM+I} = component due to vertical effects (i.e., end rotations of spans at structural expansion joints), and
- δ_{LDBE} = component due to LDBE alone

For Group 4:

- max δ_{LF} = 0.20” [2, Section 6.5.4.5.2(1)],
- max δ_{LLRM+I} = 0.30” [2, Section 6.5.4.5.2(2)], therefore
- max $\delta_{EXP} \leq$ 0.50”

For Group 5:

- max δ_{LF} = 0.20” [2, Section 6.5.4.5.2(1)],
- max δ_{LLRM+I} = 0.30” [2, Section 6.5.4.5.2(2)],
- max δ_{LDBE} = 0.50”, therefore,
- max $\delta_{EXP} \leq$ 1.0” [4, Volume 3, Section 3.6.1.1]

These limits are for continuous welded rails (ballasted or direct fixation) without rail expansion joints [2, Section 6.5.4.5.2].

The component of relative longitudinal displacement due to vertical effects, $\delta_{LLRM+I,}$ corresponds to the following end rotation limits at expansion joints under either one or two tracks loaded [4, Volume 3, Section 3.9.1].

For ballasted and slab tracks, with long welded rails, the end rotation of the bridge due to vertical effects only, LLRM + I, at the expansion joints is limited to:

$\theta = \left[\dfrac{2.625x10^{-2}}{h(ft)}\right] radians$, for transitions between the decks and abutments

$\theta_1 + \theta_2 = \left[\dfrac{2.625x10^{-2}}{h(ft)}\right] radians$, between consecutive decks with equal superstructure heights.

$\theta_1 + \theta_2 = \left[\dfrac{1.312x10^{-2}}{h_1(ft)}\right] + \left[\dfrac{1.312x10^{-2}}{h_2(ft)}\right] radians$, between consecutive decks with different superstructure heights.

Where,

h(ft): the distance between the top of rail and the center of the bridge bearing

h_1(ft): the distance between the top of rail and the center of the first bridge bearing

h_2(ft): the distance between the top of rail and the center of the second bridge bearing

See Figure 6-6 for the definition of end rotations at expansion joints.

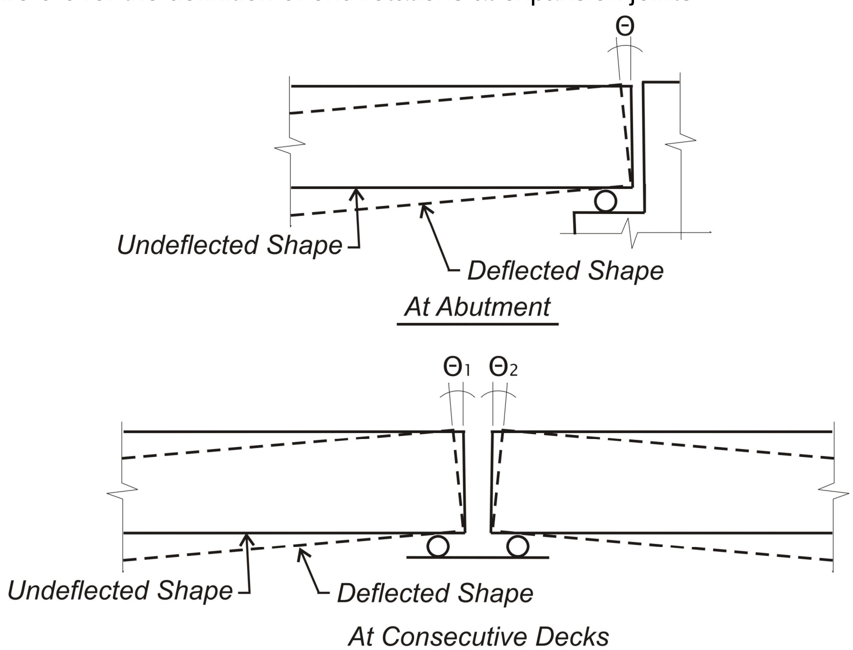


Figure 6-6: End Rotations at Expansion Joints

6.6.4 Relative Vertical Displacement at Bridge Ends

The relative vertical deck deflection, δ_V, between the bridge deck end and adjacent deck or abutment shall be limited for traffic safety.

For Group 4:

- max δ_V = 0.08" [2, Section 6.5.4.5.2(3)],

For Group 5:

- max δ_V = 0.16"

6.6.5 Uplift at Direct Fixation Rail

For Group 5 loading, where direct fixation used, the fastening system capacity shall be designed to withstand any calculated uplift force (F_{uplift}) by a factor safety of 2.

6.6.6 Rail Stress Range Limits

For rails on the bridge or aerial structure and adjacent abutments or at-grade regions, the maximum allowable rail stress range limits, including TD, are [4, Volume 3, Section 3.6.1.1]:

For Group 4f: -10.0 ksi ≤ σ ≤ 13.0 ksi

For Group 5f: -20.0 ksi ≤ σ ≤ 24.0 ksi

Where negative is compression, and positive is tension.

Note that the during evaluation of the rail stress range, any stress effects due to pre-heating at the time of construction shall be considered as per TM 2.1.5: Track Design.

These limiting values for rail stress range limits are valid for:

- Standard or high strength rail with a yield strength of at least 74.0 ksi and a tensile strength of 142.5 ksi. For project rail type recommendations, see TM 2.1.5: Track Design.
- The maximum limit from the fixed point to the free point of structure (i.e., structural thermal unit) is 330 ft.
- Straight track or track radius r ≥ 4920 ft
- Direct fixation slab track with maximum fastener spacing of 24".
- Ballasted track with heavy concrete sleepers of maximum spacing of 24" or equivalent track construction, and a minimum of 12" of consolidated ballast under the sleepers.

For rail fasteners, the non-linear force displacement characteristics in Figure 6-8 or Figure 6-9 shall be the basis for design. Should the fasteners experience plastic deformation ($\Delta_{plastic}$), then sufficient ultimate displacement capacity (Δ_{ult}) shall exist for $\Delta_{ult} > 1.5\Delta_{plastic}$, in order to ensure against fastener fracture.

6.7 DYNAMIC ANALYSIS USING ACTUAL HIGH-SPEED TRAINS

Guidelines are pending

6.8 PASSENGER COMFORT ANALYSIS

Guidelines are pending

6.9 MODELING REQUIREMENTS

6.9.1 General

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

6.9.2 Model Geometry and Boundary Conditions

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

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

For repetitive aerial structure viaducts with simply supported spans the model shall have a minimum of twenty (20) spans, with the subject span as the middle span. Boundary conditions at the ends of the model shall represent the stiffness of any adjacent spans or frames.

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

Soil springs at the foundations shall be per the Project Geotechnical Report.

6.9.3 Model Stiffness

Maximum dynamic amplification occurs at resonance, when the structure's natural vertical frequency coincides with the frequency of axle loads at resonant speed.

In order to avoid overestimating the critical resonant speed, a lower bound estimate of structure stiffness shall be included in the model.

For bridge or aerial structure piers and columns, an upper bound estimate of bending inertia of I_g, and a lower bound estimate of bending inertia of $0.8I_g$ shall be made, where I_g is the gross cross sectional bending inertia.

All structural elements shall be represented by the appropriate sectional properties and constitutive relations. Cracked bending inertias $< 0.8I_g$ for the piers and columns shall be considered, especially for LDBE response, and integral superstructure to pier connections considering cracking due to long term loadings (i.e., creep and shrinkage effects).

6.9.4 Model Mass

Both upper and lower bound estimate of bridge mass shall be considered.

In order to avoid overestimating the critical resonant speed, an upper bound estimate of mass (along with a lower bound estimate of stiffness) shall be made.

In order to avoid underestimating peak deck accelerations, a lower bound estimate of mass (along with an upper bound estimate of stiffness) shall be made.

For structural dead load (DC) mass, use the units weights given in TM 2.3.2: Structure Design Loads.

For superimposed dead load (DW), use both upper and lower bound estimates.

For ballasted track, use a lower bound estimate of ballast weight of 100 pcf at minimum ballast thickness, and an upper bound estimate of 150 pcf allowing for future ballast lifts if applicable.

6.9.5 Model Damping

The peak structural response at resonant speed is highly dependent upon damping. Table 6-4 gives the lower bound estimates of damping to be used [2, Section 6.4.6.3.1].

Table 6-4: Damping Values for Dynamic Model

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

These lower bound values may be increased for shorter spans (< 65 ft), see [2].

6.9.6 Modeling of Rail-Structure Interaction

Longitudinal actions produce longitudinal forces in the continuous rails. These forces are distributed to the bridge and aerial structures in accordance with the relative stiffness of the ties and fasteners, articulation of the structural system, and stiffness of the substructure, see Figure 6-7.

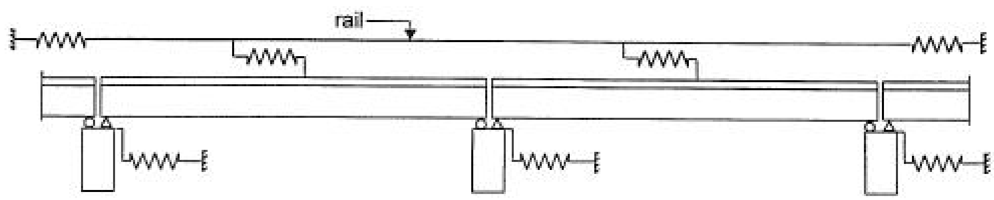


Figure 6-7: Rail-Structure Interaction Model

Rail-structure interaction is important in that it may govern the:

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

Rail-structure interaction shall be performed for all structures, using either static or dynamic models. In addition, the model shall include the axial stiffness of the rails appropriately located upon the superstructure, and longitudinal bi-linear coupling springs between the track and superstructure over the length of the model. The bi-linear coupling springs shall represent either the ballast track (see Figure 6-8) or direct fixation slab track (see Figure 6-9) between the rails and superstructure [2, Section 6.5.4.4].

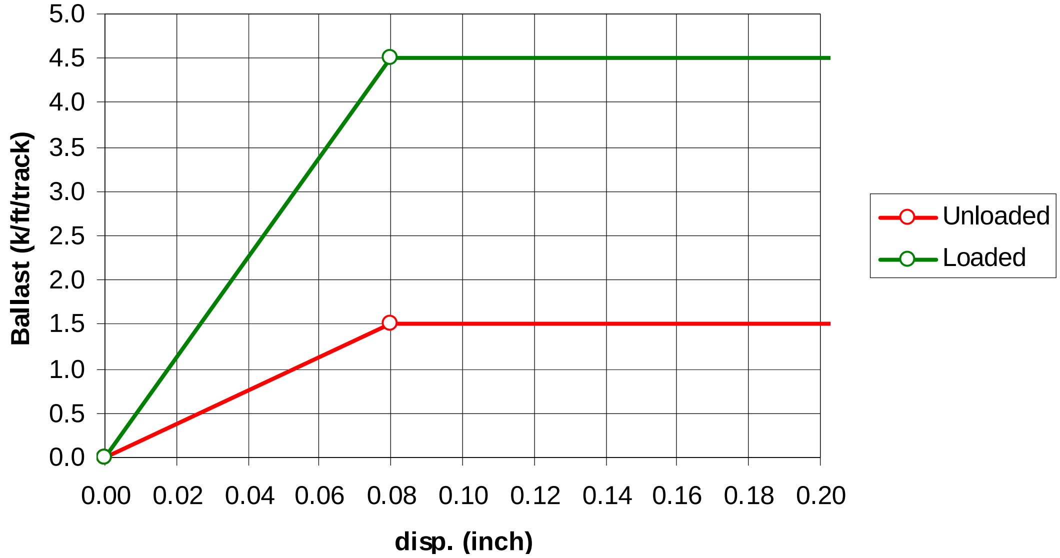


Figure 6-8: Ballasted Track Bi-linear Coupling Springs

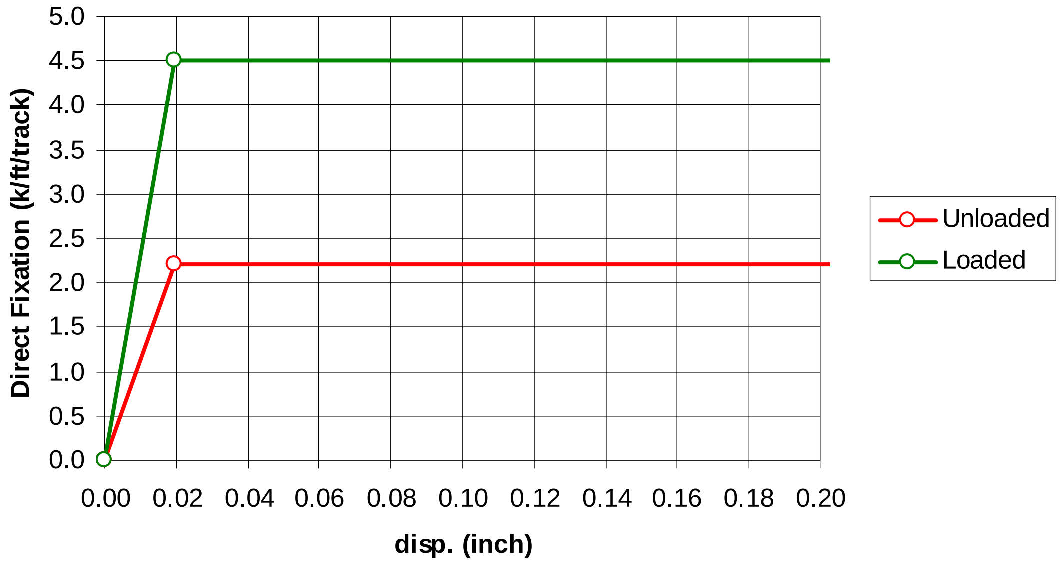


Figure 6-9: Direct Fixation Slab Track Bi-linear Coupling Springs

The total number of springs per each span shall not be less than ten (10) and the spacing between the springs shall not be more than 15 ft.

Where an abutment occurs at the viaduct ends, the rails and coupling springs shall be extended a minimum distance of L+130 ft from the face of the abutment; where L is equal to the average span length.

For a long viaduct the model shall consist of at least 20 spans for both normal operation analysis and earthquake analysis. A horizontal spring with a spring constant of 10,500 kips/ft shall be used at the "dead" end of each track. The yield load of the track end springs shall be equal to q_y*L/4 of the last adjacent span in the model, where q_y is the yield load (k/ft of track) of the rail. For example, q_y equals 1.5 k/ft of track for an unloaded ballast track [4].

6.10 OTHER PENDING ISSUES

6.10.1 General

Other issues will be addressed as the CHST criteria is developed. These other issues include guidelines for repetitive span arrangements, and guidelines for rail break. It is expected that other issues may arise in the future.

6.10.2 Guidelines for Repetitive Span Arrangements

Guidelines are pending.

6.10.3 Guidelines for Rail Break

Guidelines are pending.

6.10.4 Guidelines for Camber due to Creep/Shrinkage

Guidelines are pending.

6.10.5 Guidelines for Settlement at Supports

Guidelines are pending

6.10.6 Guidelines for Analysis of Derailment Loads

Guidelines are pending

6.10.7 Guidelines for Application of Nosing and Hunting Forces

Guidelines are pending