

California High-Speed Train Project



TECHNICAL MEMORANDUM

15% Seismic Design Benchmarks TM 2.10.5

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Revision	Date	Description
0	15 Mar 10	Initial Release, R0. Issued for 15% Design

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



For the California High-Speed Rail Authority

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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.

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Systems:	<u>NOT REQUIRED</u> Print Name:	_____ Date
Infrastructure:	<u>Signed document on file</u> Roupen Donikian, PE	<u>06 Mar 10</u> Date
Operations:	<u>NOT REQUIRED</u> Print Name:	_____ Date
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ABSTRACT

The California High-Speed Train Project (CHSTP) will provide high-speed train service within the state of California, between the San Francisco Bay Area and Sacramento to Los Angeles and south to San Diego. The high-speed train alignment passes through some of the most seismically active regions of California, including crossings of major fault systems.

This Technical Memorandum provides benchmark guidance to document the scope, confirm design feasibility, establish the structure foundation footprint, ensure constructability, and develop preliminary cost estimates for the 15% Design level and to support the EIR/EIS documents. Project specific structure design is to be presented in the form of a Caltrans Advance Planning Study (APS) that will support the required environmental documents. The structural seismic design and analysis standards, appropriate for the 30% Design and Final Design levels are defined in TM 2.10.4 Interim Seismic Design Criteria.

These benchmark guidelines shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, tunnels and underground structures, passenger stations and buildings. The level of 15% seismic design is based upon structural technical classification as defined in TM 2.10.4:

- For structures technically classified as “standard” or “non-standard”, no seismic design is required for 15% unless foundations may interfere with existing structures or facilities to remain.
- For structures classified as “complex”, Equivalent Static Analysis (ESA) for seismic loading is required in order to define the foundation footprints, verify structural framing feasibility, and provide preliminary construction cost estimates.

At the 15% Design level, limited project-specific seismic and geotechnical information is available. In the absence of complete information, recommendations are given herein for seismic and geotechnical input assumptions in order to advance the 15% Design.

1.0 INTRODUCTION

1.1 PURPOSE OF TECHNICAL MEMORANDUM

The seismic structural design and analysis standards, appropriate for the 30% Design and Final Design levels are defined in TM 2.10.4 Interim Seismic Design Criteria. However, for the project-level environmental assessment stage (15% Design) a different set of requirements is warranted.

This technical memorandum provides benchmark guidance to assist in establishing the scope, confirming design feasibility, establishing the structure foundation footprint, ensuring reasonable constructability, and developing preliminary cost estimates for EIR/EIS documents. Project specific structure design is to be presented in the form of a Caltrans Advance Planning Study (APS) that will support the required environmental documents.

These benchmark guidelines shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, tunnels and underground structures, passenger stations and buildings

1.2 STATEMENT OF TECHNICAL ISSUE

The technical issue is to provide regional design teams with guidance for seismic design in support of 15% Design. For subsequent design levels, TM 2.10.4 Interim Seismic Design Criteria shall be used.

1.3 GENERAL INFORMATION

1.3.1 Definition of Terms

The following 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
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
APS	Advance Planning Study
ASCE	American Society of Civil Engineers
CBDA	Caltrans Bridge Design Aids Manual
CBDD	Caltrans Bridge Design Details Manual
CBDP	Caltrans Bridge Design Practice Manual
CBDS	Caltrans Bridge Design Specifications
CBC	California Building Code
CBDM	California Department of Transportation (Caltrans) Bridge Design Manuals, Latest Edition
CDC	CHST Project Design Criteria
CHSTP	California High-Speed Train Project
CMTD	Caltrans Bridge Memo to Designers Manual
CSDC	Caltrans Seismic Design Criteria
D	Dead load
DBE	Design Basis Earthquake
ESA	Equivalent Static Analysis
FHWA	Federal Highway Administration
LDBE	Lower-level Design Basis Earthquake
LRFD	Load and Resistance Factor Design
MCE	Maximum Considered Earthquake
NCL	No Collapse Performance Level
OCS	Overhead Contact System
OPL	Operability Performance Level
PGA	Peak Ground Acceleration

SPL	Safety Performance Level
SSI	Soil-structure Interaction
TM	Technical Memorandum
USGS	United States Geologic Survey
Δ_D	Displacement Demand
Δ_C	Displacement Capacity

1.3.2 Units

The California High-Speed Train Project is based on U.S. Customary Units consistent with guidelines prepared by the California Department of Transportation and defined by the National Institute of Standards and Technology (NIST). U.S. Customary Units are officially used in the United States, and are also known in the U.S. as “English” or “Imperial” units. In order to avoid confusion, all formal references to units of measure shall be made in terms of U.S. Customary Units. In some cases, U.S. Customary Units are not applicable as some of the analytical equations require inputs in SI units. In those cases, only SI units are mentioned.

2.0 DEFINITION OF TECHNICAL TOPIC

2.1 15% SEISMIC DESIGN REQUIREMENTS BASED UPON TECHNICAL CLASSIFICATIONS

These seismic benchmark guidelines shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, tunnels and underground structures, passenger stations and buildings.

Based on a structure's technical classification, as defined in TM 2.10.4 Interim Seismic Design Criteria: Section 6.1.1.3, the 15% seismic design requirements are as follows:

- For structures classified as “standard” or “non-standard, no seismic design is required for the 15% Design milestone unless foundations interfere with existing structures or facilities to remain. If foundations potentially interfere, then an Equivalent Static Analysis (ESA), as defined in TM 2.10.4 Interim Seismic Design, Section 6.5.4.2, shall be used to define the foundation footprint.
- For structures classified as “complex”, ESA for the Maximum Considered Earthquake (MCE) (as defined in TM 2.10.4: Section 6.2.3) are required in order to establish foundation footprints, verify structural framing feasibility, and provide preliminary cost estimates.

The objective of the guidelines is to ensure that complex structures under consideration are feasible and will meet the “No Collapse” performance level under the MCE event at the 15% Design level of project development.

Per TM 2.10.4: Section 6.1.1.3, “complex” structures are defined. Examples of complex structural features include:

- **Irregular Geometry** - Structures that include multiple superstructure levels, variable width or bifurcating superstructures, highly skewed supports, or support columns of drastically varying height.
- **Unusual Framing** - Structures that include outrigger or C-bent supports, unbalanced mass and/or stiffness distribution, extremely tall support columns, or multiple superstructure types.
- **Long Aerial Structure Spans** - Aerial structures with spans greater than 300 feet.
- **Unusual Geologic Conditions** - Structures that are founded on soft soil, soil having moderate to high liquefaction potential, soil of significantly varying type over the length of the structure, or structures located in close proximity to earthquake faults. Unusual geologic conditions and near source seismic effects will be defined by the Geotechnical Report.

For 15% seismic design, all tunnels shall be considered “complex”.

Per TM 2.10.4: Section 6.2.3, the MCE is defined according to ASCE 7-05 [9]. The design ground motions probabilistically correspond to having a 4% probability of exceedance within the design life of 100 years (return period of about 2,475 years), and deterministically are within the limits as explained in ASCE 7-05 [9, Section 21.2.2].

3.0 ASSESSMENT / ANALYSIS

At the 15% Design level, limited project specific seismic and geotechnical information will be available. In the absence of project-specific information, recommendations are given below for seismic and geotechnical input assumptions in order to advance the 15% Design.

3.1 15% DESIGN MCE SPECTRA

3.1.1 Elevated Structures

In the absence of any project specific seismic design spectra, designers are directed to United States Geological Survey (USGS) Earthquake Hazards Program. USGS has developed 2002 United States National Seismic Hazard Maps of contours of seismic design parameters for the MCE event. These mapped parameters are accessible through USGS earthquake ground motion Java calculator [14], which may be used during 15% seismic design.

For details regarding the seismic ground motion parameters, refer to ASCE7-05 [9, Section 11.4], which defines the parameters S_{MS} and S_{M1} : the 0.2-second and 1.0-second site adjusted spectral response acceleration for the MCE-type event, respectively. The default spectral damping is 5%.

Within the Java calculator, 15% Design MCE spectra are to be developed as follows:

- Choose ASCE 7 Standard.
- Input project specific latitude and longitude.
- If no soil data is available, assume Site Class D.
- Establish site modified S_a vs. T spectral ordinates for MCE.
- Multiply the Site Modified S_a vs. T spectral ordinates for MCE by an Importance Factor, $I = 1.25$ (i.e., Occupancy Category III per ASCE 7-05).

Based upon the structure's fundamental period in each orthogonal direction, the static multiplier, S_a , is determined from linear interpolation between the given horizontal spectral ordinates, and used in the ESA for the MCE.

The peak MCE ground vertical seismic load effect is assumed to be $0.2IS_{MS}D$, where D is the tributary dead load.

3.1.2 Tunnels

For preliminary design of tunnels, peak particle acceleration and velocity at depth can be estimated based upon the modal moment magnitude, M_w , of earthquake and source to site distance (Section 3.4.6). This information is not directly available through ASCE 7-05 methodology.

If site-specific seismological information is not available, designers are directed to the USGS 2008 Interactive Deaggregation website [15], which uses 2008 United States National Seismic Hazard Maps [16], and the Next Generation of Attenuation models [24, 25, 26, 27, 28].

The method to determine seismic design parameters for tunnel design from the Deaggregation website is as follows:

- Input project specific latitude and longitude.
- Select return period of 2% in 50 years (for 2475 year return period)
- If no soil data is available, assume shear wave velocity in upper 30m of soil (V_{s30}) of 300 m/s, consistent with Site Class D per the 2007 CBC [7].
- Find the Peak Ground Acceleration (PGA) (i.e., spectral acceleration at $T = 0.0s$).
- Determine the PGA (g), modal moment magnitude (M_w), and source to site distance (km) from the deaggregation results.

- Compare the PGA from the deaggregation results to the PGA from ASCE 7-05 method. If reasonably close ($\pm 10\%$), then use the PGA from ASCE 7-05 results, the modal moment magnitude (M_w) and source to site distance (km) from the deaggregation results, to estimate peak particle acceleration and velocity at depth (Section 3.4.6).
- The estimated peak particle acceleration and velocity at depth are used to determine tunnel demands from closed form solutions given in Section 3.4.

3.2 GEOTECHNICAL ASSUMPTIONS FOR 15% LEVEL DESIGN

Once foundation demands are found based upon the ESA, preliminary foundation sizing can occur. In lieu of site-specific geotechnical information, the following geotechnical assumptions may be made for 15% seismic design. These assumptions are only for 15% Design in order to estimate foundation size, and will be superseded once site-specific geotechnical data are available during future design levels. These assumptions are intended to be generally conservative in order to give conservative estimates of foundation sizes. These are to be used in LRFD limit state evaluation of foundations for the MCE event [2, Section 10: Foundations].

3.2.1 Assumed Spread Footing Strength Limit Values

For spread footing foundations, assume a strength limit state bearing pressure $Q_n = 6000$ psf, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_n = 6000$ psf). Footings to be embedded a minimum of 5 feet below grade.

Assume a strength limit state passive pressure $Q_{ep} = 800$ pcf (equivalent fluid weight), with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_{ep} = 800$ pcf). Neglect the top 2 feet of embedment for passive pressure resistance.

Assume a strength limit state internal friction angle of soil, $\Phi_f = 30^\circ$, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_r = 1.0 P_{down} \tan \Phi_f$, where P_{down} is the total vertically downwards force).

Overturning and sliding stability checks are to be performed for spread footing foundations [2, Section 10].

3.2.2 Assumed Pile or Drilled Shaft Strength Limit Values

For pile or drilled shaft foundations, consistent with Site Class D, assume the following strength limit state skin friction (in absence of self-weight):

Download, Q_d (psf)	Upload, Q_u (psf)
1200	600

with a resistance factor, $\Phi = 1.0$ for downloads (i.e., $\Phi Q_d = 1200$ psf for downloads) and $\Phi = 0.8$ for uploads, $\Phi Q_u = 480$ psf for uploads). Pile caps shall be assumed to be embedded a minimum of 5 feet below grade.

The assumed strength limit state for pile or drilled shaft passive pressure $Q_{ep} = 1000$ pcf (equivalent fluid weight) acting over 1.5 diameters, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_{ep} = 1000$ pcf). The passive pressure on the embedded portion of the pile caps shall be as per Section 3.2.1 above.

Pile or drilled shaft group effects shall be considered.

3.3 15% SEISMIC DESIGN - ELEVATED STRUCTURES

The seismic design of elevated structures discussed in this technical memorandum shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, passenger stations and buildings.

As an approximate approach for the preliminary design of the elevated structures, ESA shall be used. At 15% Design, only structures classified as "complex" (per TM 2.10.4: Section 6.2.3) need to be seismically evaluated.

3.3.1 Design Philosophy

For elevated structures, the intended structural performance under seismic loading is that of a Limited Ductility Structure, whereby:

- Elevated structures shall have a clearly defined mechanism for response to seismic loads with a clearly defined load path and load carrying system.
- For bridges and aerial structures, the inelastic behavior shall be limited to columns, piers and abutments.
- For stations and buildings, a “weak beam - strong column” approach shall be used.

In general, the designer allows specified structural components to undergo inelastic behavior under MCE by providing a fusing mechanism, while force-protecting other components. Current Caltrans seismic analysis and design philosophies as stated in Caltrans Bridge Design Manuals (CBDM) [8] form the basis of the 15% Design.

3.3.2 Equivalent Static Seismic Analysis – Load Case

Equivalent Static Analysis (ESA) for the MCE is required for design of “complex” structures.

From TM: 2.3.2: Structure Design Loads [11], Table 6.4-1, the MCE load case classification is “Extreme 3”. For vertical loads the analysis shall consider the seismic vertical component (i.e., $\pm 0.2S_aD$).

3.3.3 Allowable Strain Limits

As per TM 2.10.4 Interim Seismic Design Criteria: Section 6.5.5.1.2.1, allowable strain limits for the reinforced concrete design (i.e., for no-collapse performance under MCE event) are:

- Mild reinforcing steel tensile allowable strain limits (ϵ_{su}^a):

$$\epsilon_{su}^a \leq 2/3 \epsilon_{su}$$
, where ϵ_{su} is the ultimate tensile strain of reinforcing steel,
- Concrete confined compressive allowable strain limit (ϵ_{cu}^a):

$$\epsilon_{cu}^a \leq 2/3 \epsilon_{cu}$$
, where ϵ_{cu} is the ultimate confined compressive strain as computed by Mander’s model for confined concrete,
- Concrete unconfined compressive allowable strain limits (ϵ_{cu}^a):

$$\epsilon_{cu}^a = 0.004$$

The unconfined compressive strain is to be applied to concrete members without sufficient lateral reinforcement to be considered confined. If the lateral reinforcement does not meet the requirements of CBDM for confinement, the section should be considered unconfined. There are no requirements for the unconfined concrete cover.

As per TM 2.10.4 Interim Seismic Design Criteria: Section 6.5.5.1.2.2, allowable strain limits for the structural steel design (i.e., for no-collapse performance under MCE event) are:

$$\epsilon_{su}^a \leq 2/3 \epsilon_{su}$$
, where ϵ_{su} is the ultimate tensile strain of steel

3.3.4 Structural Frequencies

In order to meet train and passenger comfort performance criteria, structures supporting high-speed trains are expected to be very rigid and stiff, resulting in natural vibration modes in the high frequency range.

Regional designers are directed to TM 2.10.10: High-Speed Train and Track Structure Compatibility [13] for required structural frequency limits for train-structure compatibility. These frequency limits will be a significant factor in establishing the required stiffness of the structural systems.

3.3.5 Displacement Demand ($C\Delta_D$)

Where ESA is required, the standard procedures contained in Caltrans Seismic Design Criteria (CSDC) [8] shall apply. Longitudinal and transverse “stand-alone” analysis shall be used, with

fixed base boundary conditions (i.e., no foundation flexibility) and required effective sectional properties per Caltrans Bridge Memo to Designers Manual (CMTD) section 20-4 and CSDC section 5.6.

The equivalent static shear force to be used in the ESA is the product of the static multiplier, S_a as determined from Section 3.1.1, and the tributary dead load (D), applied at the superstructure's center of mass. Applying this to the stand-alone models will result in the horizontal displacement demand, Δ_D , of the superstructure's center of mass.

Per TM 2.10.4, the displacement demand, Δ_D , obtained from ESA shall be multiplied by an amplification factor, C, as follows:

$$\text{For } T/T_o < 1: \quad C = [0.8 / (T/T_o)] + 0.2$$

$$\text{For } T/T_o > 1: \quad C = 1.0$$

where,

T = fundamental period of structure

T_o = the period centered on the peak of the acceleration response spectrum.

3.3.6 Displacement Capacity (Δ_C)

The horizontal displacement capacity, Δ_C , shall be determined by nonlinear static displacement capacity or "pushover" analysis. The displacement capacity shall be defined as the controlling structure displacement that occurs when any primary element reaches its specified capacity in the pushover analysis. Primary flexural members shall be checked to ensure that they have sufficient shear capacity to ensure formation of ductile mechanisms.

For the fixed base assumption, the displacement capacity evaluation presented in CSDC section 3.1 shall be followed. This will require a limited evaluation of local member curvature capacity, which is subject to the allowable reinforcement and concrete strains, presented in TM 2.10.4, Section 6.5.5.1.2.1 for the MCE event. The assumptions made to determine the displacement capacity, Δ_C , shall be consistent with those used to determine the displacement demand, $C\Delta_D$.

3.3.7 Displacement Demand versus Capacity Ratio

The maximum displacement Demand/Capacity Ratio shall be $C\Delta_D / \Delta_C \leq 1.0$.

3.3.8 Capacity Protected Design

In order to limit inelasticity to prescribed ductile elements, the plastic moments and shears of the ductile elements shall be used in the demand/capacity analysis of the non-ductile, capacity-protected elements of the structure.

Component over-strength design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC. For the case of concrete columns, the over-strength factor for foundation design shall be 1.2 (i.e., design moment for foundation is 20% higher than the plastic moment capacity of the column).

3.3.9 15% Design level Design of Foundations

Based upon the over-strength moment, plastic shear, and axial loading at each column, the foundations shall be sized using the assumed geotechnical values given in Section 3.2.

Care shall be taken to verify that the foundation footprint does not interfere with existing structures or facilities to remain.

3.3.10 Verifying Structural Framing Feasibility

For complex structures having unusual framing conditions, such as outrigger or C-bent support, the 15% Design shall verify that such framing is feasible from both a structural and construction viewpoint. Issues such as outrigger support articulation (i.e., "pinned" support at the top of the column), or large torsional demands on outrigger or C-bents shall be investigated and the conclusions reflected in the cost estimate.

3.4 15% SEISMIC DESIGN - TUNNELS

The seismic design of tunnels discussed in this technical memorandum shall include bored tunnels, mined tunnels, submerged tubes and cut-and-cover tunnels.

Generally, seismic response of tunnels is dominated by the surrounding ground response, and not the inertial properties of the tunnel itself. The focus of tunnel seismic design shall be on the free-field deformation of the surrounding ground and its interaction with the tunnel.

For 15% Design, two types of deformations which characterize the seismic response of tunnels shall be evaluated:

1. Longitudinal axial and curvature deformations (see **Figure 3-1**)
2. Transverse ovaling or racking deformation (see **Figure 3-3**)

The Federal Highway Administration (FHWA) has created an online technical manual [17, <http://www.fhwa.dot.gov/bridge/tunnel/pubs/nhi09010/13a.cfm>] which summarizes approximate and simplified closed-form procedures for these deformations. These procedures shall be used to evaluate the 15% seismic design of tunnels, with regard to deformation and strain demands.

The simplified procedures to determine longitudinal axial and curvatures deformation demands draw mainly upon the work of St. John and Zahrah [18].

The simplified procedures to determine transverse ovaling or racking deformation demands draw mainly upon the work of Wang [19] and Penzien [20].

Other source information may be found in the works of Power [21], and Hashash [22, 23].

The applicability of these procedures is limited, and therefore, a more detailed numerical approach shall be used for later levels of design.

There are three exceptions to the preliminary evaluation approach described in this section:

1. Geological or soil conditions change drastically over a short distance of the tunnel alignment.
2. Tunnel alignment crosses a known seismic fault.
3. Tunnel alignment proposed in an area subject to ground failure (i.e., liquefaction or slope instability).

If any of these conditions exist, more detailed numerical analysis, as given in TM 2.10.4, shall be performed to demonstrate that the design can accommodate these special conditions.

3.4.1 Design Philosophy

The intended structural performance of tunnels under seismic loading shall meet the design requirements of Limited Ductility Structures, whereby:

- Tunnels or underground structures shall have a clearly defined mechanism for response to seismic loads with a clearly defined load path and load carrying system.
- Inelastic behavior shall be limited to designated regions, where ductility is provided using appropriate detailing procedures. The remainder of the structure is capacity protected to prevent brittle failure mechanisms and instability.

Limited and targeted inelasticity under the MCE shall be used for the design of tunnels, while force-protecting other components. Component over-strength design shall follow CSDC, with an overstrength factor of 1.2 (i.e., the design for capacity protected members shall be 20% higher than the inelastic response). This over-strength capacity assures the desired ductile mechanism occurs and that the undesired non-ductile failure mechanisms are prevented from forming.

3.4.2 Equivalent Static Seismic Analysis – Load Case

Equivalent Static Seismic Analysis (ESA) for the MCE is required for design of tunnels and buried structures considered “complex” structures.

From TM: 2.3.2: Structure Design Loads, Table 6.4-1, the MCE load case is “Extreme 3”.

3.4.3 Allowable Strain Limits

For tunnels, the same allowable strain limits per elevated structures apply (Section 3.3.3).

3.4.4 Longitudinal Axial and Curvature Deformations

Longitudinal axial and curvature deformations occur in tunnels when seismic waves propagate either parallel or obliquely to the tunnel. The axial and curvature induced longitudinal strains in the tunnel lining shall be evaluated based upon the procedures given in ref. [17, 18].

Figure 3-1 shows the idealized representations of the axial and curvature deformations. The general behavior is that of an elastic beam subject to imposed ground deformations.

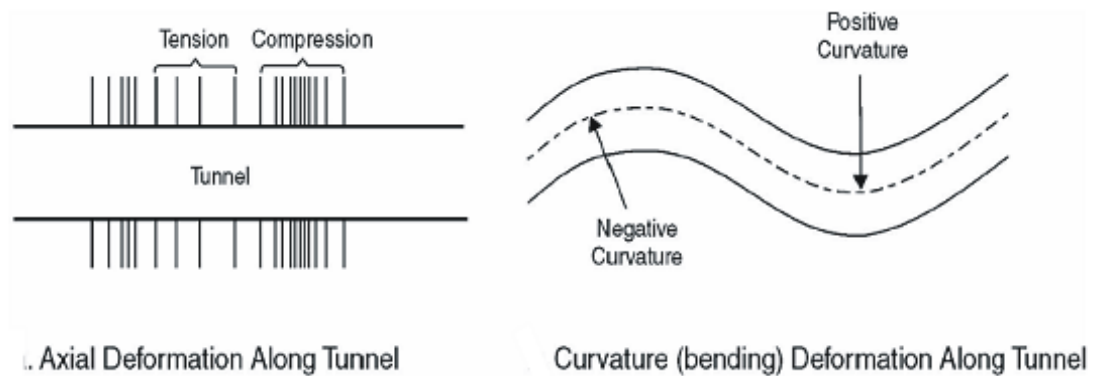


Figure 3-1: Longitudinal Axial and Curvature Deformations [17, 19]

Figure 3-2 shows the idealized sectional forces due to axial and curvature deformations.

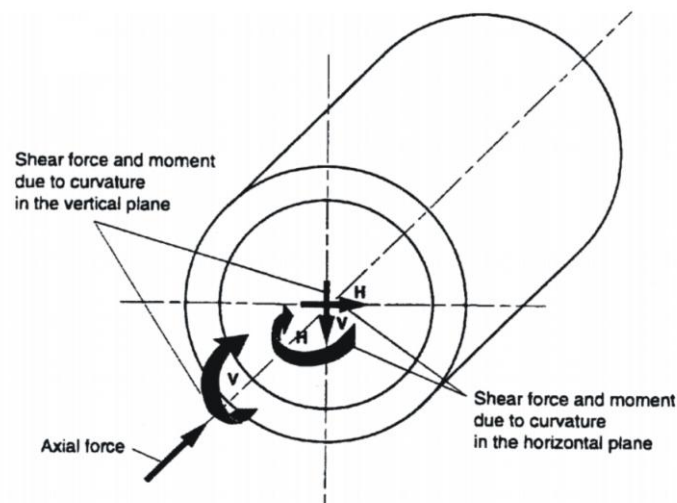


Figure 3-2: Sectional Forces Due to Curvature and Axial Deformations [21]

3.4.5 Transverse Ovaling and Racking Deformations

Transverse ovaling and racking deformations occur in tunnel structures when seismically induced vertically propagating shear waves cause ground movement in the laterally transverse direction. The deformations and strains due to these motions, which result in tunnel cross-sectional distortion as shown in **Figure 3-3**, shall be evaluated based upon the procedures outlined in ref. [17, 19, 20].

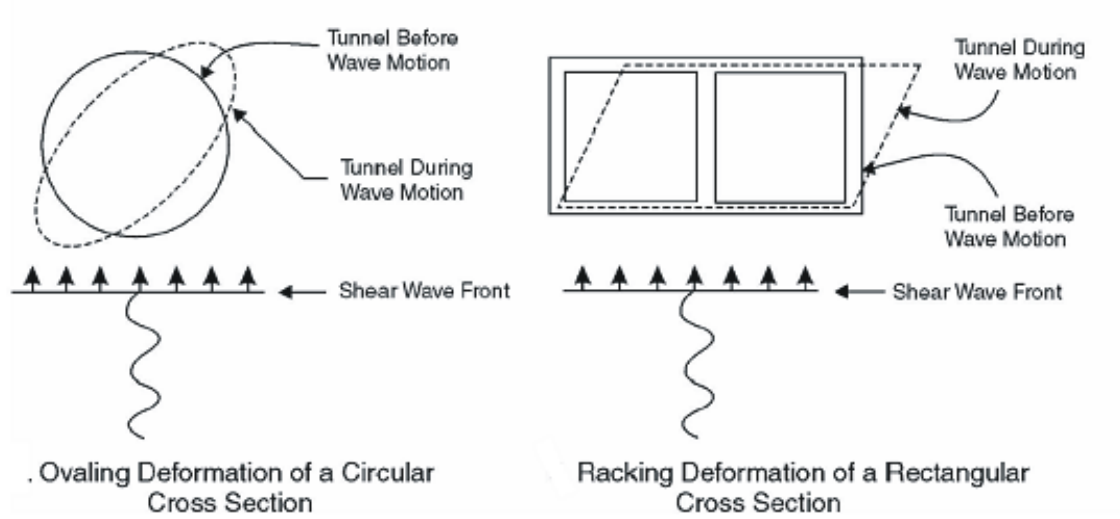


Figure 3-3: Transverse Ovaling and Racking [17, 19]

Figure 3-4 shows the idealized circumferential forces acting on the “through thickness” of the tunnel, due to ovaling and racking deformations. The “no-slip” assumption (i.e., full connection between surrounding soils and tunnel lining) contained in ref. [19] shall be used to evaluate tunnel lining thrust.

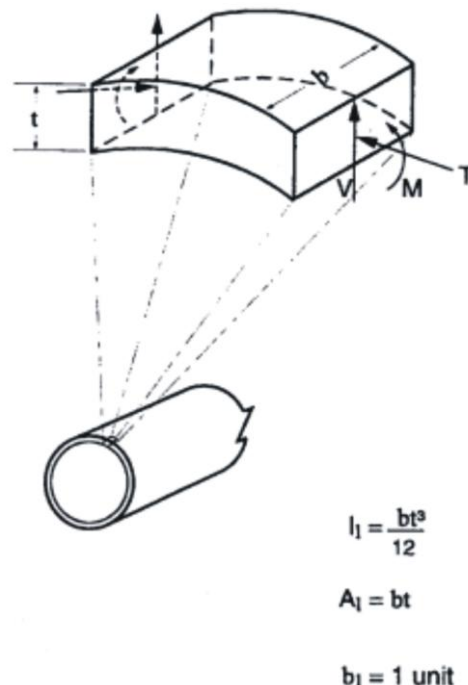


Figure 3-4: Circumferential Forces and Moments [21]

3.4.6 Estimation of Peak Soil Particle Acceleration and Velocity

For the closed-form procedures above, peak soil particle acceleration (a_s) and velocity (V_s) are needed to determine the seismic demands.

Peak particle acceleration (a_s) at tunnel depth, which is generally less than ground surface acceleration, shall be determined based upon **Table 3-1**, which gives the reduction factors as a function of tunnel depth. The peak ground acceleration (PGA) shall be obtained from the site specific 15% MCE spectra (Section 3.1).

Table 3-1: Ratio of peak particle acceleration at depth to PGA at surface [21]

Tunnel Depth (m)	Ratio of peak particle acceleration at depth to PGA at surface
≤ 6m	1.0
6 to 15 m	0.9
15 to 30 m	0.8
> 30 m	0.7

Peak particle velocity (V_s) shall be estimated based upon the peak particle acceleration, moment magnitude of earthquake (M_w), and source to site distance. For 15% Design, the modal moment magnitude earthquake and source to site distance obtained from the USGS deaggregation analysis shall be used (Section 3.1), along with the conversion factors given in **Table 3-2**.

Table 3-2: Ratio of peak particle velocity (cm/s) to peak particle acceleration (g) in rock and soil [21]

Moment Magnitude, M_w	Source to site distance (km)		
	0 to 20	20 to 50	50 to 100
Rock			
6.5	66	76	86
7.5	97	109	97
8.5	127	140	152
Stiff Soil			
6.5	94	102	109
7.5	140	127	155
8.5	180	188	193
Soft Soil			
6.5	140	132	142
7.5	208	165	201
8.5	269	244	251

Note that in **Table 3-2**:

- Rock is defined as having shear wave velocity, $C_s \geq 750$ m/s,
- Stiff soil is defined as having shear wave velocity, 200 m/s $< C_s < 750$ m/s
- Soft soil is defined as having shear wave velocity, $C_s < 200$ m/s

3.4.7 Proximity Analysis

When determining the alignment, the clear distance between a proposed tunnel and an existing structure or a second proposed tunnel (twin bore) is a critical parameter to be investigated. During a seismic event adequate clearance must be maintained such that a proposed tunnel will not adversely affect another structure.

For the 15% Design level, a proximity analysis shall be performed whenever the clear distance between an existing structure and the proposed tunnel is within two diameters of the proposed tunnel. For twin bore tunnel configurations, the pillar width shall be evaluated to provide adequate clearance.

3.5 15% CONSTRUCTION COST ESTIMATES

Construction cost estimates for the 15% Design are to be supported by estimated quantities or area factors for major items of work, consistent with Caltrans Bridge Design Aids Manual (CBDA) Chapter 11 Estimating for Planning Estimates [8].

Special or restrictive construction requirements shall be reflected in the cost estimate. The mitigation costs associated with special seismic conditions such as liquefaction and fault crossings shall be considered when developing cost estimates.

3.6 ADVANCED PLANNING STUDY SUPPORT

The main goal of 15% Design is to support the APS and the development of subsequent environmental documents.

Based upon APS requirements, the following general plans shall be developed:

- Structural plan views
- Structural elevations
- Typical structural cross sections

The structural plan view shall show the basic structural layout including horizontal alignment, existing structures and facilities to be removed or to remain, location of critical overhead and underground utilities, and foundation footprints.

The structural elevations shall show preliminary vertical profile grades, structural depths, and critical clearance requirements.

The typical structural cross sections shall show the proposed structure sized appropriately for the expected HST application, and include the train tracks, ballast (if used), Overhead Contact System (OCS) poles, walkways and barriers, with the required horizontal clearances for train travel.

The APS shall delineate any special or restrictive construction requirements, including limited site accessibility, seasonal work limitations, or expected work within existing right of ways. Also, a construction or detour plan shall be developed, if needed, for any roadways or railways which require on-going functionality during construction.

4.0 SUMMARY AND RECOMMENDATIONS

At the 15% Design level, detailed project specific seismic and geotechnical information is not available. Lacking this information, specific recommendations have been given for seismic and geotechnical input in order to advance 15% Design.

The level of 15% seismic design needed is based upon structure's technical classification. Only "complex" structures, as defined in TM: 2.10.4: Interim Seismic Design Criteria, require analysis at the 15% Design. This analysis is used to define the foundation footprints, verify structural framing feasibility, and provide preliminary construction cost estimates.

Benchmark guidance to support 15% seismic design for high-speed train bridges, aerial structures, tunnels and underground structures, passenger stations and buildings is presented in Section 6.0.

Project-specific 15% Design shall be prepared in accordance with Advance Planning Studies (APS) guidelines set forth in Caltrans Memo to Designers and Office of Special Funded Projects Information and Procedures Guide [8] manuals.

5.0 SOURCE INFORMATION AND REFERENCES

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 - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification, 4th Edition, 2007, with California Amendments.
 - Caltrans Bridge Memo to Designers Manual (CMTD)
 - Caltrans Bridge Design Practices Manual (CBPD)
 - Caltrans Bridge Design Aids Manual (CBDA)
 - Caltrans Bridge Design Details Manual (CBDD)
 - Caltrans Standard Specifications
 - Caltrans Standard Plans
 - Caltrans Seismic Design Criteria ver. 1.4 (CSDC)
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6.0 DESIGN MANUAL CRITERIA

6.1 15% SEISMIC DESIGN

The guidance in this technical memorandum is not intended to be a part of the CHSTP Design Manual. The purpose of this technical memorandum is to define a minimum level of engineering design required to confirm design feasibility for environmental assessment and prepare a preliminary construction cost estimate.

6.1.1 15% Seismic Design Requirements

At the 15% Design level, detailed project specific seismic and geotechnical information is not available. Lacking this information, specific recommendations have been given for seismic and geotechnical input in order to advance 15% Design.

The level of 15% seismic design needed is based upon structure's technical classification. Only "complex" structures, as defined in TM: 2.10.4: Interim Seismic Design Criteria, require analysis at the 15% Design. This analysis is used to define the foundation footprints, verify structural framing feasibility, and provide preliminary construction cost estimates.

Project-specific 15% Design shall be prepared in accordance with Advance Planning Studies (APS) guidelines set forth in Caltrans Memo to Designers and Office of Special Funded Projects Information and Procedures Guide [8] manuals.

6.1.2 Technical Classifications

These seismic benchmark guidelines shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, tunnels and underground structures, passenger stations and buildings.

Based on a structure's technical classification, as defined in TM 2.10.4 Interim Seismic Design Criteria: Section 6.1.1.3, the 15% seismic design requirements are as follows:

- For structures classified as "standard" or "non-standard, no seismic design is required for the 15% Design milestone unless foundations interfere with existing structures or facilities to remain. If foundations potentially interfere, then an Equivalent Static Analysis (ESA), as defined in TM 2.10.4 Interim Seismic Design, Section 6.5.4.2, shall be used to define the foundation footprint.
- For structures classified as "complex", ESA for the Maximum Considered Earthquake (MCE) (as defined in TM 2.10.4: Section 6.2.3) are required in order to establish foundation footprints, verify structural framing feasibility, and provide preliminary cost estimates.

The objective of the guidelines is to ensure that complex structures under consideration are feasible and will meet the "No Collapse" performance level under the MCE event at the 15% Design level of project development.

Per TM 2.10.4: Section 6.1.1.3, "complex" structures are defined. Examples of complex structural features include:

- **Irregular Geometry** - Structures that include multiple superstructure levels, variable width or bifurcating superstructures, highly skewed supports, or support columns of drastically varying height.
- **Unusual Framing** - Structures that include outrigger or C-bent supports, unbalanced mass and/or stiffness distribution, extremely tall support columns, or multiple superstructure types.
- **Long Aerial Structure Spans** - Aerial structures with spans greater than 300 feet.
- **Unusual Geologic Conditions** - Structures that are founded on soft soil, soil having moderate to high liquefaction potential, soil of significantly varying type over the length of the structure, or structures located in close proximity to earthquake faults.

Unusual geologic conditions and near source seismic effects will be defined by the Geotechnical Report.

For 15% seismic design, all tunnels shall be considered “complex”.

Per TM 2.10.4: Section 6.2.3, the MCE is defined according to ASCE 7-05 [9]. The design ground motions probabilistically correspond to having a 4% probability of exceedance within the design life of 100 years (return period of about 2,475 years), and deterministically are within the limits as explained in ASCE 7-05 [9, Section 21.2.2].

6.2 15% DESIGN MCE SPECTRA

6.2.1 Elevated Structures

In the absence of any project specific seismic design spectra, designers are directed to United States Geological Survey (USGS) Earthquake Hazards Program. USGS has developed 2002 United States National Seismic Hazard Maps of contours of seismic design parameters for the MCE event. These mapped parameters are accessible through USGS earthquake ground motion Java calculator [14], which may be used during 15% seismic design.

For details regarding the seismic ground motion parameters, refer to ASCE7-05 [9, Section 11.4], which defines the parameters S_{MS} and S_{M1} : the 0.2-second and 1.0-second site adjusted spectral response acceleration for the MCE-type event, respectively. The default spectral damping is 5%.

Within the Java calculator, 15% Design MCE spectra are to be developed as follows:

- Choose ASCE 7 Standard.
- Input project specific latitude and longitude.
- If no soil data is available, assume Site Class D.
- Establish site modified S_a vs. T spectral ordinates for MCE.
- Multiply the Site Modified S_a vs. T spectral ordinates for MCE by an Importance Factor, $I = 1.25$ (i.e., Occupancy Category III per ASCE 7-05).

Based upon the structure's fundamental period in each orthogonal direction, the static multiplier, S_a , is determined from linear interpolation between the given horizontal spectral ordinates, and used in the ESA for the MCE.

The peak MCE ground vertical seismic load effect is assumed to be $0.2IS_{MS}D$, where D is the tributary dead load.

6.2.2 Tunnels

For preliminary design of tunnels, peak particle acceleration and velocity at depth can be estimated based upon the modal moment magnitude, M_w , of earthquake and source to site distance (Section 6.5.6). This information is not directly available through ASCE 7-05 methodology.

If site-specific seismological information is not available, designers are directed to the USGS 2008 Interactive Deaggregation website [15], which uses 2008 United States National Seismic Hazard Maps [16], and the Next Generation of Attenuation models [24, 25, 26, 27, 28].

The method to determine seismic design parameters for tunnel design from the Deaggregation website is as follows:

- Input project specific latitude and longitude.
- Select return period of 2% in 50 years (for 2475 year return period)
- If no soil data is available, assume shear wave velocity in upper 30m of soil (V_{s30}) of 300 m/s, consistent with Site Class D per the 2007 CBC [7].
- Find the Peak Ground Acceleration (PGA) (i.e., spectral acceleration at $T = 0.0s$).

- Determine the PGA (g), modal moment magnitude (M_w), and source to site distance (km) from the deaggregation results.
- Compare the PGA from the deaggregation results to the PGA from ASCE 7-05 method. If reasonably close ($\pm 10\%$), then use the PGA from ASCE 7-05 results, the modal moment magnitude (M_w) and source to site distance (km) from the deaggregation results, to estimate peak particle acceleration and velocity at depth (Section 6.5.6).
- The estimated peak particle acceleration and velocity at depth are used to determine tunnel demands from closed form solutions given in Section 6.5.

6.3 GEOTECHNICAL ASSUMPTIONS FOR 15% LEVEL DESIGN

Once foundation demands are found based upon the ESA, preliminary foundation sizing can occur. In lieu of site-specific geotechnical information, the following geotechnical assumptions may be made for 15% seismic design. These assumptions are only for 15% Design in order to estimate foundation size, and will be superseded once site-specific geotechnical data are available during future design levels. These assumptions are intended to be generally conservative in order to give conservative estimates of foundation sizes. These are to be used in LRFD limit state evaluation of foundations for the MCE event [2, Section 10: Foundations].

6.3.1 Assumed Spread Footing Strength Limit Values

For spread footing foundations, assume a strength limit state bearing pressure $Q_n = 6000$ psf, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_n = 6000$ psf). Footings to be embedded a minimum of 5 feet below grade.

Assume a strength limit state passive pressure $Q_{ep} = 800$ pcf (equivalent fluid weight), with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_{ep} = 800$ pcf). Neglect the top 2 feet of embedment for passive pressure resistance.

Assume a strength limit state internal friction angle of soil, $\Phi_f = 30^\circ$, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_r = 1.0 P_{down} \tan \Phi_f$, where P_{down} is the total vertically downwards force).

Overtopping and sliding stability checks are to be performed for spread footing foundations [2, Section 10].

6.3.2 Assumed Pile or Drilled Shaft Strength Limit Values

For pile or drilled shaft foundations, consistent with Site Class D, assume the following strength limit state skin friction (in absence of self-weight):

Download, Q_d (psf)	Upload, Q_u (psf)
1200	600

with a resistance factor, $\Phi = 1.0$ for downloads (i.e., $\Phi Q_d = 1200$ psf for downloads) and $\Phi = 0.8$ for uploads, $\Phi Q_u = 480$ psf for uploads). Pile caps shall be assumed to be embedded a minimum of 5 feet below grade.

The assumed strength limit state for pile or drilled shaft passive pressure $Q_{ep} = 1000$ pcf (equivalent fluid weight) acting over 1.5 diameters, with a resistance factor, $\Phi = 1.0$ (i.e., $\Phi Q_{ep} = 1000$ pcf). The passive pressure on the embedded portion of the pile caps shall be as per Section 6.3.1.

Pile or drilled shaft group effects shall be considered.

6.4 15% SEISMIC DESIGN - ELEVATED STRUCTURES

The seismic design of elevated structures discussed in this technical memorandum shall be applied to all structures that directly support track and running high-speed trains including bridges, aerial structures, passenger stations and buildings.

As an approximate approach for the preliminary design of the elevated structures, ESA shall be used. At 15% Design, only structures classified as "complex" (per TM 2.10.4: Section 6.2.3) need to be seismically evaluated.

6.4.1 Design Philosophy

For elevated structures, the intended structural performance under seismic loading is that of a Limited Ductility Structure, whereby:

- Elevated structures shall have a clearly defined mechanism for response to seismic loads with a clearly defined load path and load carrying system.
- For bridges and aerial structures, the inelastic behavior shall be limited to columns, piers and abutments.
- For stations and buildings, a “weak beam - strong column” approach shall be used.

In general, the designer allows specified structural components to undergo inelastic behavior under MCE by providing a fusing mechanism, while force-protecting other components. Current Caltrans seismic analysis and design philosophies as stated in Caltrans Bridge Design Manuals (CBDM) [8] form the basis of the 15% Design.

6.4.2 Equivalent Static Seismic Analysis – Load Case

Equivalent Static Analysis (ESA) for the MCE is required for design of “complex” structures.

From TM: 2.3.2: Structure Design Loads [11], Table 6.4-1, the MCE load case classification is “Extreme 3”. For vertical loads the analysis shall consider the seismic vertical component (i.e., $\pm 0.2S_aD$).

6.4.3 Allowable Strain Limits

As per TM 2.10.4 Interim Seismic Design Criteria: Section 6.5.5.1.2.1, allowable strain limits for the reinforced concrete design (i.e., for no-collapse performance under MCE event) are:

- Mild reinforcing steel tensile allowable strain limits (ϵ_{su}^a):

$$\epsilon_{su}^a \leq 2/3 \epsilon_{su}$$
, where ϵ_{su} is the ultimate tensile strain of reinforcing steel,
- Concrete confined compressive allowable strain limit (ϵ_{cu}^a):

$$\epsilon_{cu}^a \leq 2/3 \epsilon_{cu}$$
, where ϵ_{cu} is the ultimate confined compressive strain as computed by Mander’s model for confined concrete,
- Concrete unconfined compressive allowable strain limits (ϵ_{cu}^a):

$$\epsilon_{cu}^a = 0.004$$

The unconfined compressive strain is to be applied to concrete members without sufficient lateral reinforcement to be considered confined. If the lateral reinforcement does not meet the requirements of CBDM for confinement, the section should be considered unconfined. There are no requirements for the unconfined concrete cover.

As per TM 2.10.4 Interim Seismic Design Criteria: Section 6.5.5.1.2.2, allowable strain limits for the structural steel design (i.e., for no-collapse performance under MCE event) are:

$$\epsilon_{su}^a \leq 2/3 \epsilon_{su}$$
, where ϵ_{su} is the ultimate tensile strain of steel

6.4.4 Structural Frequencies

In order to meet train and passenger comfort performance criteria, structures supporting high-speed trains are expected to be very rigid and stiff, resulting in natural vibration modes in the high frequency range.

Designers are directed to TM 2.10.10: High-Speed Train and Track Structure Compatibility [13] for required structural frequency limits for train-structure compatibility. These frequency limits will be a significant factor in establishing the required stiffness of the structural systems.

6.4.5 Displacement Demand ($C\Delta_D$)

Where ESA is required, the standard procedures contained in Caltrans Seismic Design Criteria (CSDC) [8] shall apply. Longitudinal and transverse “stand-alone” analysis shall be used, with fixed base boundary conditions (i.e., no foundation flexibility) and required effective sectional properties per Caltrans Bridge Memo to Designers Manual (CMTD) section 20-4 and CSDC section 5.6.

The equivalent static shear force to be used in the ESA is the product of the static multiplier, S_a as determined from Section 6.2.1, and the tributary dead load (D), applied at the superstructure’s center of mass. Applying this to the stand-alone models will result in the horizontal displacement demand, Δ_D , of the superstructure’s center of mass.

Per TM 2.10.4, the displacement demand, Δ_D , obtained from ESA shall be multiplied by an amplification factor, C, as follows:

$$\text{For } T/T_o < 1: \quad C = [0.8 / (T/T_o)] + 0.2$$

$$\text{For } T/T_o > 1: \quad C = 1.0$$

where,

T = fundamental period of structure

T_o = the period centered on the peak of the acceleration response spectrum.

6.4.6 Displacement Capacity (Δ_C)

The horizontal displacement capacity, Δ_C , shall be determined by nonlinear static displacement capacity or “pushover” analysis. The displacement capacity shall be defined as the controlling structure displacement that occurs when any primary element reaches its specified capacity in the pushover analysis. Primary flexural members shall be checked to ensure that they have sufficient shear capacity to ensure formation of ductile mechanisms.

For the fixed base assumption, the displacement capacity evaluation presented in CSDC section 3.1 shall be followed. This will require a limited evaluation of local member curvature capacity, which is subject to the allowable reinforcement and concrete strains, presented in TM 2.10.4, Section 6.5.5.1.2.1 for the MCE event. The assumptions made to determine the displacement capacity, Δ_C , shall be consistent with those used to determine the displacement demand, $C\Delta_D$.

6.4.7 Displacement Demand versus Capacity Ratio

The maximum displacement Demand/Capacity Ratio shall be $C\Delta_D / \Delta_C \leq 1.0$.

6.4.8 Capacity Protected Design

In order to limit inelasticity to prescribed ductile elements, the plastic moments and shears of the ductile elements shall be used in the demand/capacity analysis of the non-ductile, capacity-protected elements of the structure.

Component over-strength design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC. For the case of concrete columns, the over-strength factor for foundation design shall be 1.2 (i.e., design moment for foundation is 20% higher than the plastic moment capacity of the column).

6.4.9 15% Design level Design of Foundations

Based upon the over-strength moment, plastic shear, and axial loading at each column, the foundations shall be sized using the assumed geotechnical values given in Section 6.3.

Care shall be taken to verify that the foundation footprint does not interfere with existing structures or facilities to remain.

6.4.10 Verifying Structural Framing Feasibility

For complex structures having unusual framing conditions, such as outrigger or C-bent support, the 15% Design shall verify that such framing is feasible from both a structural and construction viewpoint. Issues such as outrigger support articulation (i.e., “pinned” support at the top of the

column), or large torsional demands on outrigger or C-bents shall be investigated and the conclusions reflected in the cost estimate.

6.5 15% SEISMIC DESIGN - TUNNELS

The seismic design of tunnels discussed in this technical memorandum shall include bored tunnels, mined tunnels, submerged tubes and cut-and-cover tunnels.

Generally, seismic response of tunnels is dominated by the surrounding ground response, and not the inertial properties of the tunnel itself. The focus of tunnel seismic design shall be on the free-field deformation of the surrounding ground and its interaction with the tunnel.

For 15% Design, two types of deformations which characterize the seismic response of tunnels shall be evaluated:

3. Longitudinal axial and curvature deformations (see **Figure 6-1**)
4. Transverse ovaling or racking deformation (see **Figure 6-3**)

The Federal Highway Administration (FHWA) has created an online technical manual [17, <http://www.fhwa.dot.gov/bridge/tunnel/pubs/nhi09010/13a.cfm>] which summarizes approximate and simplified closed-form procedures for these deformations. These procedures shall be used to evaluate the 15% seismic design of tunnels, with regard to deformation and strain demands.

The simplified procedures to determine longitudinal axial and curvatures deformation demands draw mainly upon the work of St. John and Zahrah [18].

The simplified procedures to determine transverse ovaling or racking deformation demands draw mainly upon the work of Wang [19] and Penzien [20].

Other source information may be found in the works of Power [21], and Hashash [22, 23].

The applicability of these procedures is limited, and therefore, a more detailed numerical approach shall be used for later levels of design.

There are three exceptions to the preliminary evaluation approach described in this section:

4. Geological or soil conditions change drastically over a short distance of the tunnel alignment.
5. Tunnel alignment crosses a known seismic fault.
6. Tunnel alignment proposed in an area subject to ground failure (i.e., liquefaction or slope instability).

If any of these conditions exist, more detailed numerical analysis, as given in TM 2.10.4, shall be performed to demonstrate that the design can accommodate these special conditions.

6.5.1 Design Philosophy

The intended structural performance of tunnels under seismic loading shall meet the design requirements of Limited Ductility Structures, whereby:

- Tunnels or underground structures shall have a clearly defined mechanism for response to seismic loads with a clearly defined load path and load carrying system.
- Inelastic behavior shall be limited to designated regions, where ductility is provided using appropriate detailing procedures. The remainder of the structure is capacity protected to prevent brittle failure mechanisms and instability.

Limited and targeted inelasticity under the MCE shall be used for the design of tunnels, while force-protecting other components. Component over-strength design shall follow CSDC, with an overstrength factor of 1.2 (i.e., the design for capacity protected members shall be 20% higher than the inelastic response). This over-strength capacity assures the desired ductile mechanism occurs and that the undesired non-ductile failure mechanisms are prevented from forming.

6.5.2 Equivalent Static Seismic Analysis – Load Case

Equivalent Static Seismic Analysis (ESA) for the MCE is required for design of tunnels and buried structures considered “complex” structures.

From TM: 2.3.2: Structure Design Loads, Table 6.4-1, the MCE load case is “Extreme 3”.

6.5.3 Allowable Strain Limits

For tunnels, the same allowable strain limits per elevated structures apply (Section 6.4.3).

6.5.4 Longitudinal Axial and Curvature Deformations

Longitudinal axial and curvature deformations occur in tunnels when seismic waves propagate either parallel or obliquely to the tunnel. The axial and curvature induced longitudinal strains in the tunnel lining shall be evaluated based upon the procedures given in ref. [17, 18].

Figure 6-1 shows the idealized representations of the axial and curvature deformations. The general behavior is that of an elastic beam subject to imposed ground deformations.

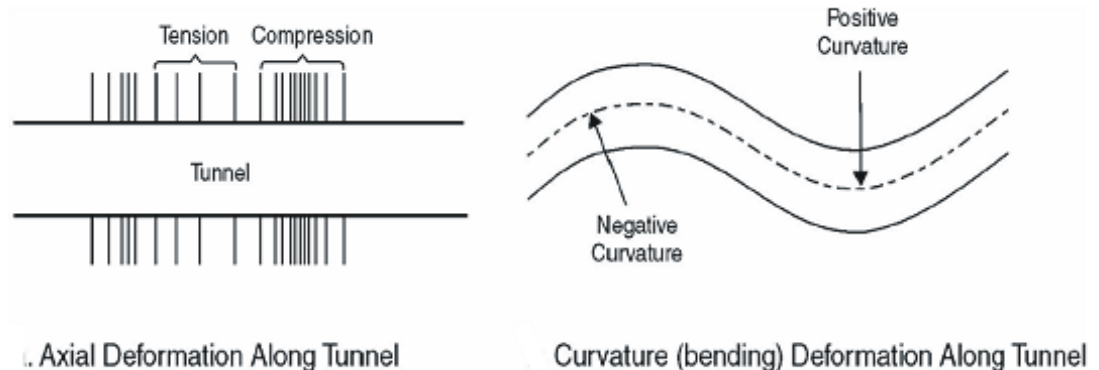


Figure 6-1: Longitudinal Axial and Curvature Deformations [17, 19]

Figure 6-2 shows the idealized sectional forces due to axial and curvature deformations.

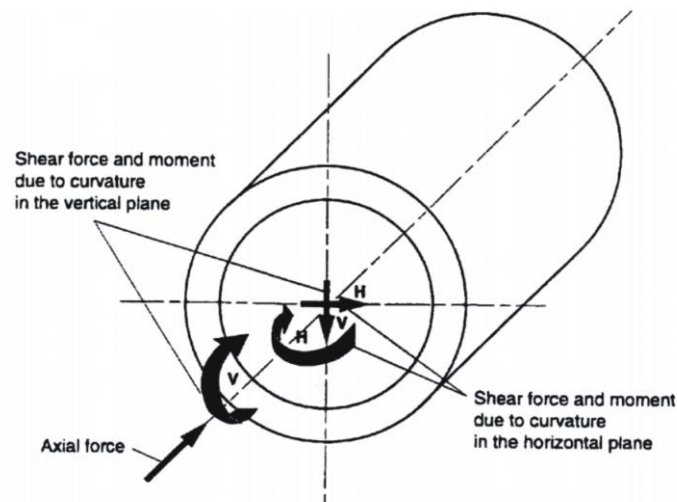


Figure 6-2: Sectional Forces Due to Curvature and Axial Deformations [21]

6.5.5 Transverse Ovaling and Racking Deformations

Transverse ovaling and racking deformations occur in tunnel structures when seismically induced vertically propagating shear waves cause ground movement in the laterally transverse direction. The deformations and strains due to these motions, which result in tunnel cross-sectional distortion as shown in **Figure 6-3**, shall be evaluated based upon the procedures outlined in ref. [17, 19, 20].

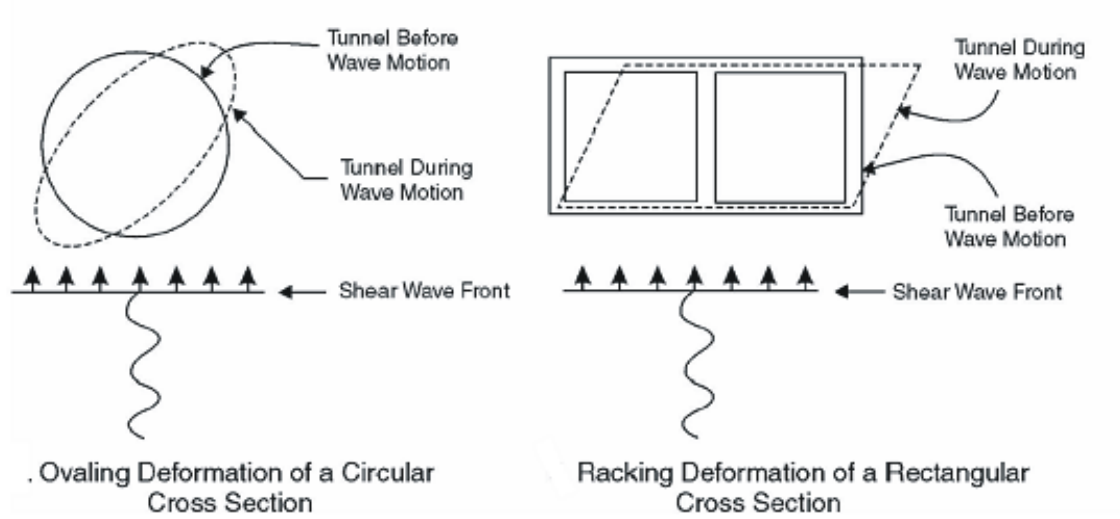


Figure 6-3: Transverse Ovaling and Racking [17, 19]

Figure 6-4 shows the idealized circumferential forces acting on the “through thickness” of the tunnel, due to ovaling and racking deformations. The “no-slip” assumption (i.e., full connection between surrounding soils and tunnel lining) contained in ref. [19] shall be used to evaluate tunnel lining thrust.

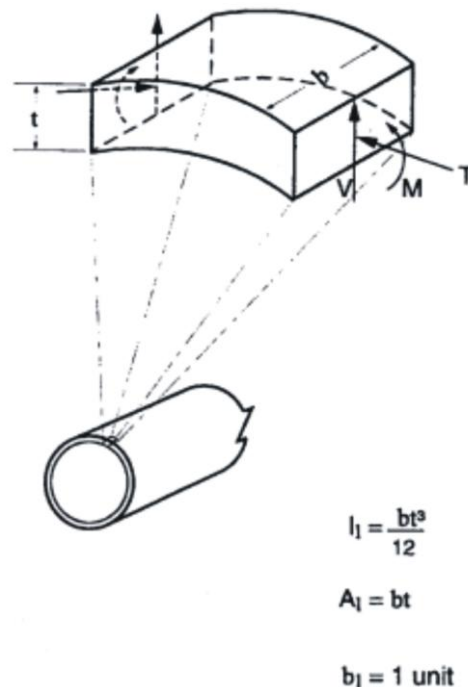


Figure 6-4: Circumferential Forces and Moments [21]

6.5.6 Estimation of Peak Soil Particle Acceleration and Velocity

For the closed-form procedures above, peak soil particle acceleration (a_s) and velocity (V_s) are needed to determine the seismic demands.

Peak particle acceleration (a_s) at tunnel depth, which is generally less than ground surface acceleration, shall be determined based upon **Table 6-1**, which gives the reduction factors as a function of tunnel depth. The peak ground acceleration (PGA) shall be obtained from the site specific 15% MCE spectra (Section 6.2).

Table 6-1: Ratio of peak particle acceleration at depth to PGA at surface [21]

Tunnel Depth (m)	Ratio of peak particle acceleration at depth to PGA at surface
≤ 6m	1.0
6 to 15 m	0.9
15 to 30 m	0.8
> 30 m	0.7

Peak particle velocity (V_s) shall be estimated based upon the peak particle acceleration, moment magnitude of earthquake (M_w), and source to site distance. For 15% Design, the modal moment magnitude earthquake and source to site distance obtained from the USGS deaggregation analysis shall be used (Section 6.2), along with the conversion factors given in **Table 6-2**:

Table 6-2: Ratio of peak particle velocity (cm/s) to peak particle acceleration (g) in rock and soil [21]

Moment Magnitude, M_w	Source to site distance (km)		
	0 to 20	20 to 50	50 to 100
Rock			
6.5	66	76	86
7.5	97	109	97
8.5	127	140	152
Stiff Soil			
6.5	94	102	109
7.5	140	127	155
8.5	180	188	193
Soft Soil			
6.5	140	132	142
7.5	208	165	201
8.5	269	244	251

Note that:

- Rock is defined as having shear wave velocity, $C_s \geq 750$ m/s,
- Stiff soil is defined as having shear wave velocity, 200 m/s $< C_s < 750$ m/s
- Soft soil is defined as having shear wave velocity, $C_s < 200$ m/s

6.5.7 Proximity Analysis

When determining the alignment, the clear distance between a proposed tunnel and an existing structure or a second proposed tunnel (twin bore) is a critical parameter to be investigated. During a seismic event adequate clearance must be maintained such that a proposed tunnel will not adversely affect another structure.

For the 15% Design level, a proximity analysis shall be performed whenever the clear distance between an existing structure and the proposed tunnel is within two diameters of the proposed tunnel. For twin bore tunnel configurations, the pillar width shall be evaluated to provide adequate clearance.

6.6 15% CONSTRUCTION COST ESTIMATES

Construction cost estimates for the 15% Design are to be supported by estimated quantities or area factors for major items of work, consistent with Caltrans Bridge Design Aids Manual (CBDA) Chapter 11 Estimating for Planning Estimates [8].

Special or restrictive construction requirements shall be reflected in the cost estimate. The mitigation costs associated with special seismic conditions such as liquefaction and fault crossings shall be considered when developing cost estimates.

6.7 ADVANCED PLANNING STUDY SUPPORT

The main goal of 15% Design is to support the APS and the development of subsequent environmental documents.

Based upon APS requirements, the following general plans shall be developed:

- Structural plan views
- Structural elevations
- Typical structural cross sections

The structural plan view shall show the basic structural layout including horizontal alignment, existing structures and facilities to be removed or to remain, location of critical overhead and underground utilities, and foundation footprints.

The structural elevations shall show preliminary vertical profile grades, structural depths, and critical clearance requirements.

The typical structural cross sections shall show the proposed structure sized appropriately for the expected HST application, and include the train tracks, ballast (if used), Overhead Contact System (OCS) poles, walkways and barriers, with the required horizontal clearances for train travel.

The APS shall delineate any special or restrictive construction requirements, including limited site accessibility, seasonal work limitations, or expected work within existing right of ways. Also, a construction or detour plan shall be developed, if needed, for any roadways or railways which require on-going functionality during construction.