California High-Speed Train Project

TECHNICAL MEMORANDUM

Seismic Design Criteria

Structures Supporting High-Speed Trains
TM 2.10.4

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

Prepared by PB

for the California High-Speed Rail Authority

System Level Technical and Integration Reviews

The purpose of the review is to ensure:

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

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

System Level Technical Reviews by Subsystem:

Systems:	NOT REQUIRED Rick Schmedes	DD Month YY Date
Infrastructure:	*Signed document on file* Tom Jackson	23 June 11 Date
Operations:	NOT REQUIRED Joseph Metzler	DD Month YY Date
Maintenance:	NOT REQUIRED Joseph Metzler	DD Month YY Date
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Note: Signatures apply for the technical memorandum revision corresponding to revision number in header and as noted on cover

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ABSTRACT

The California High-Speed Train Project (CHSTP) will provide high-speed train service within the state of California, linking the San Francisco Bay Area and Sacramento in the north, to Los Angeles and San Diego in the south. The high-speed train alignment will pass through regions of high seismic activity, including crossings of major fault systems.

This Technical Memorandum (TM) establishes seismic design criteria and guidance for structures directly supporting high-speed train service, including but not limited to, bridges, aerial structures, tunnels, underground structures, stations, and building structures. These structures, defined as Primary Structures, shall be designed according to this TM.

It is necessary to establish policy on the seismic retrofit of existing and new structures owned by other entities, not directly supporting high-speed train service, but having the potential to impact high-speed train service. This policy decision is pending.

Secondary structures, those not supporting or potentially impacting, high-speed train service, shall be designed according to TM 2.5.1: Structural Design of Surface Facilities and Buildings.

This TM defines structural classifications, seismic performance objectives and requirements, acceptable damage, relevant design codes/standards, acceptable methodologies and procedures, and design criteria.

For 15% design, TM 2.10.5: 15% Seismic Design Benchmarks shall apply.

For 30% and final design, this TM shall apply.

1.0 INTRODUCTION

1.1 PURPOSE OF TECHNICAL MEMORANDUM

This Technical Memorandum (TM) establishes seismic design criteria and guidance for structures supporting high-speed train service, including but not limited to, bridges, aerial structures, tunnels, underground structures, stations, and building structures. These structures are defined as Primary structures.

Secondary structures, those not supporting, or potentially impacting, high-speed train service, shall be designed according to TM 2.5.1: Structural Design of Surface Facilities and Buildings.

This Technical Memorandum shall be used in conjunction with the following Technical Memoranda:

- TM 2.3.2: Structure Design Loads
- TM 2.5.1: Structural Design of Surface Facilities and Buildings
- TM 2.9.2: Geotechnical Reports Preparation Guidelines
- TM 2.9.3: Geologic and Seismic Hazard Analysis Guidelines
- TM 2.9.6: Interim Ground Motion Guidelines
- TM 2.9.10: Geotechnical Design Guidelines
- TM 2.10.6: Fault Rupture Analysis and Mitigation
- TM 2.10.10: Track-Structure Interaction

For seismic design criteria for earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10: Geotechnical Design Guidelines.

For 15% seismic design, TM 2.10.5: 15% Seismic Design Benchmarks shall apply.

For 30% and final design, this TM shall apply.

1.2 STATEMENT OF TECHNICAL ISSUE

This Technical Memorandum (TM) establishes seismic design criteria and guidance for Primary Structures supporting high-speed train service.

Guidelines are presented to predict demands and capacities on structures. Recommendations are provided for structural performance evaluation relative to the performance objectives and acceptable damage.

1.3 GENERAL INFORMATION

1.3.1 Definition of Terms

The following acronyms and abbreviations used in this document have specific connotations with regard to the CHSTP.

Acronyms/Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction, Manual of Steel Construction
ASCE	American Society of Civil Engineers
AWS	Structural Welding Standards

BART	Bay Area Rapid Transit
C	Amplification Factor on $\triangle_D$ for ESA and RSA
CBDA	Caltrans Bridge Design Aids Manual
CBDD	Caltrans Bridge Design Details Manual
CBDM	Caltrans Bridge Design Manuals
CBDP	Caltrans Bridge Design Practice Manual
CBDS	Caltrans Bridge Design Specifications: AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments
CBC	California Building Code 2010
CHST	California High-Speed Train
CHSTP	California High-Speed Train Project
CMTD	Caltrans Bridge Memo to Designers Manual
CQC	Complete Quadratic Combination
CSDC	Caltrans Seismic Design Criteria ver. 1.6
E	Earthquake Demands
E_L	Longitudinal Earthquake Demands
E_T	Transverse Earthquake Demands
ESA	Equivalent Static Analysis
FBE	Functional Basis Earthquake
FPL	Functional Performance Level
F_u	Elastic Force Demands including OBE events
MCE	Maximum Considered Earthquake
NCL	No Collapse Performance Level
NDP	Nonlinear Dynamic Procedure
NEHRP	National Earthquake Hazards Reduction Program
OBE	Operating Basis Earthquake
OPL	Operability Performance Level
PCF	Pounds per cubic foot
PSF	Pounds per square foot
SRSS	Square Root Sum of the Squares
SSI	Soil-structure interaction
THSR	Taiwan High Speed Rail
TM	Technical Memorandum
$\triangle_{\mathbf{D}}$	Displacement demand
$\triangle_C$	Displacement capacity
ε_{cc}	Strain at maximum compressive stress as computed by Mander's model for confined concrete
ε_{cu}	Ultimate compressive strain as computed by Mander's model for confined concrete
ε_{sh}	Tensile strain at the onset of strain hardening of steel
ε_{su}	Ultimate tensile strain of steel
ε_{ye}	Expected yield tensile strain of steel

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.

2.0 DEFINITION OF TECHNICAL TOPIC

2.1 GENERAL

This Technical Memorandum establishes seismic design criteria and guidance for Primary Structures which support high-speed train service as defined in Section 2.6.1.

A policy decision is pending as to whether new or existing structures not directly supporting high-speed train service, but having the potential to impact high-speed train service, are considered Primary structures. Further seismic design guidance will be provided for these type structures, once the policy decision has been finalized.

2.2 POLICY CONSIDERATIONS

Policy considerations regarding seismic design can significantly influence operation, risk, performance, and cost of high-speed train facilities. In developing this document, design and performance assumptions were made that will require confirmation based on Authority policy. Identified policy considerations and the assumed approach to address these issues are summarized in the following sections. Policy assumptions are liable to change.

The following policy decisions form the basis of this TM:

- Primary structures, directly supporting high-speed trains, shall be subject to this TM.
- Secondary structures, not supporting or potentially impacting high-speed trains, shall be subject to TM 2.5.1: Structural Design of Surface Facilities and Buildings.
- Regions of the high-speed train alignment may be defined as more critical than other regions. Such regions will be assigned a higher Importance Classification, and may be subject to more stringent criteria.
- The design life of fixed facilities shall be 100 years.
- The CHSTP design earthquakes and performance objectives are based upon:
  - Similar criteria in Taiwan and Japan for the lower level Operating Basis Earthquake
  - Current California Department of Transportation (Caltrans) criteria for the higher level Maximum Considered Earthquake

The following policy decisions await final approval by the Authority, and will be addressed in future versions of this TM, or separate guidance:

- The “reasonable time” for structures to be closed for inspection, repair, and track realignment, after seismic events.
- The seismic retrofit of existing and new structures owned by other entities, not directly supporting high-speed train service, but having the potential to impact high-speed train service. These include, but are not limited to, highway, freight, pedestrian, or building structures which span over, or are in close proximity to high-speed train service. Close proximity is defined as a distance where collapse, failure, or falling debris from a new or existing structure may potentially impact high-speed train service. Separate guidance will be provided for these structures once the policy decision is finalized.
- The inclusion of a Functional Performance Level (FPL) to maintain train operation and protect revenue following a moderate earthquake event. This performance level is not intended to be included for 30% Design, but may be included in the Final Design.
- The inclusion of a moderate earthquake event or Functional Basis Earthquake to be used to evaluate the FPL. The severity of this event should be defined based on an assessment of risk that can be tolerated by the program. A preliminary definition for this event may be

as follows: ground motions corresponding to a probabilistic spectrum based upon an 18% in 100 year probability of exceedance (a return period of about 500 years).

2.3 CONFLICTS IN CHSTP DESIGN CRITERIA

In the event of conflicting requirements between the CHSTP Design Criteria and other standards and codes of practice, the CHSTP Design Criteria shall take precedence. For requirements which have not been included in the CHSTP Design Criteria, the order of code precedence shall be: 1) local codes; 2) U.S. National Standards; 3) others.

Where circumstances or conflicts arise in the application of CHSTP Design Criteria, the designer shall notify the Authority or delegate for guidance. The designer shall use professional judgment during design to meet current standards of practice for seismic design of structures in California.

2.4 DESIGN VARIANCES TO SEISMIC DESIGN CRITERIA

Design variances to the seismic design criteria presented in this TM shall be made following the procedure given in TM 1.1.18: Design Variance Guidelines.

Examples of performance criteria variances include:

- Exceedance of allowable strain limits for structural components that do not meet Seismic Performance Criteria.
- Exceedance of allowable deformation limits for the track and structure or Exceedance of allowable rail stresses, under an OBE event (i.e., variance to TM 2.10.10: Track-Structure Interaction)

Examples of operational criteria variances include:

- Temporary closure for repairs following an OBE event
- Extended closures for repairs following a OBE event

Variances to CHSTP performance or operational criteria must be presented according to TM 1.1.18, and subject to review and approval by the Authority or delegate.

2.5 SEISMIC ANALYSIS AND DESIGN PLAN

The designer shall develop and submit a Seismic Analysis and Design Plan to the Authority or delegate justifying each structure's General Classification, Importance Classification, Technical Classification, and analysis techniques proposed for each structure under each design earthquake for review and approval.

The plan shall discuss the pre-determined mechanism for seismic response, including the regions subject to inelastic behavior, normally limited to columns, piers, footing foundations (i.e., rocking), and abutments. The plan shall also discuss when plastic hinging of caissons, piles, or drilled shafts is expected immediately below the soil surface for soft soil conditions.

The plan shall discuss in detail each proposed analysis, indicating the analysis software to be used as well as the modeling assumptions made and the various modeling techniques to be employed. The plan shall contain commentary as to the suitability of linear versus nonlinear analysis, considering geohazards, the severity of design ground motions, induced strains in the soil and structure, and expected nonlinearities.

The Authority or delegate will review, comment upon, and ultimately provide final approval of the Seismic Analysis and Design Plan.

2.6 STRUCTURAL CLASSIFICATIONS

CHST structures will provide a broad range of functions for the system. As such, consistent seismic design standards with different design objectives need to be applied to various structures.

Structural classification provides the method to differentiate between different seismic design objectives for the different structural types.

2.6.1 General Classifications

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

Primary Structures: Primary structures are those that directly support high-speed trains, including bridges, aerial structures, tunnels, underground structures, and stations. All primary structures are subject to the design criteria contained in this technical memorandum.

The following building structures, which are essential for high-speed train service, are considered Primary structures:

- Train control, communication, and operation control facilities
- Traction power distribution facilities
- Other equipment facilities essential for high-speed train service.

High-speed train track, track support, and rail fasteners are Primary structures. Seismic design criteria for track are given in TM 2.10.10: Track-Structure Interaction.

Earthen facilities, such as embankments, fills, retaining walls, U-walls, and reinforced soil structures, which directly support high-speed trains, are Primary structures and shall be subject to seismic design criteria as given in TM 2.9.10: Geotechnical Design Guidelines.

Secondary Structures: Secondary structures are those not supporting high-speed trains. The following structures are considered Secondary structures:

- Administrative buildings
- Shop and maintenance buildings.
- Storage facilities
- Cash handling buildings
- Parking structures
- Training facilities
- Other ancillary buildings, not essential for high-speed train service.

Secondary structures shall be subject to seismic design criteria as given in TM 2.5.1: Structural Design of Surface Facilities and Buildings.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's General Classification as Primary or Secondary. The Authority or delegate shall make the final determination on the General Classification of a structure.

2.6.2 Importance Classification

Primary structures shall be classified according to their importance. This classification will dictate the seismic performance levels the structure is required to meet.

Important Structures: Structures that are part of a critical revenue corridor as defined by the Authority or delegate.

Ordinary Structures: All structures not designated as Important are Ordinary Structures.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's Importance Classification as Important or Ordinary. The Authority or delegate will make the final determination on the Importance Classification of a structure.

2.6.3 Technical Classification

Primary structures shall be further classified according to their technical complexity as it relates to design.

Complex Structures: Structures which have complex response during seismic events are considered Complex Structures. Examples of complex structural features include:

- **Irregular Geometry -** Structures that include multiple superstructure levels, variable width or bifurcating superstructures, or adjacent frames with lateral fundamental periods of vibration varying by greater than 30%.
- **Unusual Framing -** Structures that include outrigger or C-bent supports, unbalanced mass and/or stiffness distribution, or structures with concrete columns having a ratio of height to least cross sectional dimension greater than 10 if in single curvature, and 15 if in double curvature.
- **Long Span Structures -** Structures that have spans greater than 300 feet.
- **Unusual Geologic Conditions -** Structures that are subject to unusual geologic conditions, including geologic hazards outlined in TM 2.9.3: Geologic and Seismic Hazard Guidelines. This include structures founded upon:
  - soft, collapsible, or expansive soil
  - soil having moderate to high liquefaction and other seismically induced ground deformation potential
  - soil of significantly varying type over the length of the structure.

  Unusual geologic conditions shall be defined within the Geotechnical Data Report.
- **At or in close proximity to Hazardous Faults -** For guidance for structures at or in close proximity to hazardous earthquake faults (R<20 km), see TM 2.10.6: Fault Rupture Analysis and Mitigation. Structures at or in close proximity of hazardous faults shall be designed using time history analyses including consideration of vertical earthquake motions.
- **Regions of Severe Ground Motions -** Structures located at regions where the peak ground acceleration (i.e., spectral acceleration at T=0 secs.) > 0.8 g for the Maximum Considered Earthquake (MCE).

Standard Structures: Structures that are not Complex Structures and comply with the pending CHSTP Design Guidelines for Standard Aerial Structures.

Non-Standard Structures: Structures that do not meet the requirements for Complex or Standard Structures, including structures with multiple superstructure types.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's Technical Classification as Complex, Standard, or Non-Standard. The Authority or delegate will make the final determination on the Technical Classification of a structure.

2.7 SEISMIC DESIGN POLICY

2.7.1 General

The goal of these criteria is to safeguard against loss of life, major failures, and prolonged interruption of high-speed train operations caused by structural damage due to earthquakes.

2.7.2 Seismic Performance Criteria

For structures directly supporting high-speed trains, there are three levels of Seismic Performance Criteria:

- **No Collapse Performance Level (NCL)**: Structures are able to undergo the effects of the Maximum Considered Earthquake (MCE) with no collapse. Significant damage may occur which requires extensive repair or complete replacement of some components. Occupants not on trains are able to evacuate safely. Damage and collapse due to train derailment is mitigated through containment design. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely.

- **Functional Performance Level (FPL):** Structures are able to undergo the effects of the Functional Basis Earthquake (FBE) with repairable damage and temporary service suspension. Occupants not on trains are able to evacuate safely. Damage and collapse due to train derailment is mitigated through containment design. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely. Structural damage shall be minimal, and normal service can resume within a reasonable time frame (determination pending). Short term repairs to structure and track components are expected.

  [The inclusion of the Functional Performance Level (FPL) is pending review by the Authority. This performance level is intended to protect revenue following a moderate earthquake event and will not apply during 30% Design, but may be included during Final Design depending upon the decision of the Authority.]

- **Operability Performance Level (OPL):** Structures are able to withstand the effects of the Operating Basis Earthquake (OBE) with elastic response with no spalling, and response within structural deformations limits as given in TM 2.10.10: Track-Structure Interaction, in order to limit rail stresses and protect against derailment. No derailment occurs, trains are able to safely brake from the maximum design speed to a safe stop, passengers and operators are able to evacuate stopped trains safely. Minimal disruption of service for all systems supporting high-speed train operation. Resumption of train operation within a few hours and possibly at reduced speeds.

See Table 2-1, Table 2-2, and Table 2-3 for performance objectives and acceptable damage for No Collapse Performance Level (NCL) Functional Performance Level (FPL), and Operability Performance Level (OPL), respectively.

Table 2-1: Performance Objectives/Acceptable Damage for No Collapse Performance Level (NCL)

Performance Level	Performance Objectives	Acceptable Damage
No Collapse Performance Level (NCL) Maximum Considered Earthquake (MCE)	**No Collapse Performance Level (NCL):** The main objective is to limit structural damage to prevent collapse during and after a Maximum Considered Earthquake (MCE). The performance objectives are: 1. No collapse. 2. Occupants not on trains able to evacuate safely. 3. Damage and collapse due to train derailment mitigated through containment design 4. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely. 5. Extensive repairs of complete replacement of some components of the system may be required before train operation may resume. 6. For underground structures, no flooding or mud inflow.	Significant yielding of reinforcement steel or structural steel. Minor fracturing of secondary and redundant steel members or rebar is permitted, with no collapse. Extensive cracking and spalling of concrete, but minimal loss of vertical load carrying capability Large permanent offsets that may require extensive repairs or complete replacement before operation may resume

Table 2-2: Performance Objectives/Acceptable Damage for Functional Performance Level (FPL)

Performance Level	Performance Objectives	Acceptable Damage
Functional Performance Level (FPL) Functional Basis Earthquake (FBE)	**Functional Performance Level (FPL):** The main objective is to limit structural damage to be repairable such that normal train operations can resume within a reasonable time following the Functional Basis Earthquake (FBE). The performance objectives are: 1. Limited structural and track damage, requiring short term repairs. 2. Occupants not on trains able to evacuate safely. 3. Damage and collapse due to train derailment mitigated through containment design 4. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely. 5. Resumption of train operation within a reasonable time. 6. Restore operation of all equipment within a reasonable time. 7. Safe performance in aftershocks. 8. Bridge piles shall not experience significant damage. Limited rocking of structures supported on spread footings. 9. For underground structures, no flooding or mud inflow	Yielding of reinforcement steel or structural steel, although replacement not necessary, Serviceability maintained after repairs. Spalling of concrete cover where access permits repair is allowed. Small permanent offsets, not permanently interfering with functionality or serviceability Flexural plastic hinging of the columns allowed as a fusing mechanism where rocking is not allowed or economically viable.

Table 2-3: Performance Objectives/Acceptable Damage for Operability Performance Level (OPL)

Performance Level	Performance Objectives	Acceptable Damage
Operability Performance Level (OPL) Operating Basis Earthquake (OBE)	**Operability Performance Level (OPL):** The main objective is for structures to withstand the effects of the Operating Basis Earthquake (OBE) elastic response with no spalling, and response within structural deformation limits as given in TM 2.10.10: Track-Structure Interaction, in order to limit rail stresses and protect against derailment. The performance objectives are: 1. No derailment, trains able to safely brake from the maximum design speed to a safe stop. 2. Occupants not on trains able to evacuate safely. 3. Train passengers and operators able to evacuate stopped trains safely. 4. Minimal disruption of service for all systems supporting high-speed train operation. 5. Resumption of train operations within a few hours and possibly at reduced speeds. 6. Safe performance in aftershocks 7. No rocking of bridge foundations 8. For underground structures, no flooding or mud inflow.	Elastic structural response, no structural damage. No spalling allowed. No track damage. Negligible permanent deformations.

2.7.3 Design Earthquakes

This criteria uses design earthquakes for which CHST facilities are to be designed to. The design earthquakes and performance levels are based upon similar criteria worldwide for high-speed trains, and current California Department of Transportation (Caltrans) standards.

Since more devastating earthquakes have a lower probability of occurrence, a probabilistic approach to defining earthquake hazard is used. The “return period” identifies the expected rate of occurrence for a level of earthquake. Additionally, deterministic methods are used to evaluate severe ground motions for the Maximum Considered Earthquake (MCE).

There are three levels of design earthquakes: the Maximum Considered Earthquake (MCE), the Functional Basis Earthquake (FBE), and the Operating Basis Earthquake (OBE) defined as:

- **Maximum Considered Earthquake (MCE**): Ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to M_{max}) of any fault in the vicinity of the structure.
- **Functional Basis Earthquake (FBE):** Ground motions corresponding to a probabilistic spectrum based upon an 18% probability of exceedance in 100 years (i.e., a return period of 500 years).

  [The applicability of the Functional Basis Earthquake (FBE) is pending review by the Authority. The final definition of this design earthquake will be determined at a later date and will not apply during 30% design, but may be included during final design depending upon the decision of the Authority.]
- **Operating Basis Earthquake (OBE**): Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

For more information about ground motions, including topics such as near source fling effects and the development of ground motion spectra and time histories, see TM 2.9.6: Interim Ground Motion Guidelines and TM 2.9.3: Geologic and Seismic Hazard Analysis Guidelines.

2.7.4 Hazardous Fault Crossings

TM 2.10.6: Fault Rupture Analysis and Mitigation presents the design methods and philosophies for structures at or near hazardous faults. Structures at or in close proximity of hazardous faults are classified as Complex structures and shall be designed using time history analyses including consideration of vertical earthquake motions.

2.7.5 Seismic Design Benchmarks for 15% and 30% Design

TM 2.10.5: 15% Seismic Design Benchmarks provides guidance for 15% design. Since limited project-specific seismic and geotechnical information will be available, TM 2.10.5 gives recommended methods and assumptions to be used in order to advance the 15% design

The level of 15% seismic design is based upon a Primary structure's Technical Classification:

- 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 technically classified as "complex", Equivalent Static Analysis (ESA) for NCL performance under MCE motions is required in order to define the foundation footprints, verify structural framing feasibility, and provide preliminary construction cost estimates.

For 30% and final design, the seismic criteria defined within this TM apply.

2.8 DESIGN REFERENCES AND CODES

This Technical Memorandum uses information drawn from the following references:

1. European Standard EN 1991-2:2003 Traffic Loads on Bridges
2. European Standard EN 1990:2002 +A1: 2005 Basis of Structural Design Annex A2 Application for Bridges
3. Taiwan High Speed Rail (THSR) Corporation Volume 9 Design Specifications: Section 1: General Design Specification and Section 3: Bridge Design Specification
4. Structural Design Criteria for Devil's Slide Tunnel: Final Lining and Portals

The provisions within this Technical Memorandum shall govern the design. Provisions in the following documents shall also be considered as guidelines when sufficient criteria are not provided by this Technical Memorandum.

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. ACI: American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318-05
3. AISC: American Institute of Steel Construction, Steel Construction Manual, 13th Edition
4. ASCE 41:Seismic Rehabilitation of Existing Structures
5. AWS D1.1/D1.1M:2008 Structural Welding Code-Steel
6. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code
7. AWS D1.8/D1.8M:2009 Structural Welding Code-Seismic Supplement
8. CBC: The 2010 California Building Code
9. California Department of Transportation (Caltrans) Bridge Design Manuals (CDBM)

- Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments.
- Bridge Memo to Designers Manual (CMTD)
- Bridge Design Practices Manual (CBPD)
- Bridge Design Aids Manual (CBDA)
- Bridge Design Details Manual (CBDD)
- Standard Specifications
- Standard Plans
- Seismic Design Criteria ver. 1.6 (CSDC)

The design codes referenced above are current as of May, 2011. Note that since the design codes will evolve during the duration of the CHSTP, design code references are subject to change at later dates.

Design shall meet all applicable portions of the general laws and regulations of the State of California and of respective local authorities.

2.9 LAWS AND CODES

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

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

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

3.0 ANALYSIS AND ASSESSMENT

3.1 SEISMIC DESIGN

This Technical Memorandum (TM) establishes seismic design criteria and guidance for structures supporting high-speed train service, including but not limited to, bridges, aerial structures, tunnels, underground structures, stations, and building structures. These structures are defined as Primary structures.

Secondary structures, those not supporting, or potentially impacting, high-speed train service, shall be designed according to TM 2.5.1: Structural Design of Surface Facilities and Buildings. For seismic design criteria for earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10: Geotechnical Design Guidelines.

For MCE and FBE events, a performance (i.e., strain and deformation) based design approach shall be used.

For OBE events, a force based design approach shall be used, structures are to respond elastically.

For OBE events, TM 2.10.10: Track-Structure Interaction contains track safety and rail-structure interaction criteria concurrent with high-speed train loading. For OBE events, due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the appropriate analysis technique for the track. For the structure, an elastic analysis is appropriate.

3.2 BRIDGES AND AERIAL STRUCTURES

All bridges and aerial structures supporting high-speed train service are Primary Structures.

3.2.1 Design Codes

For MCE and FBE events, current Caltrans performance based design methods and philosophies as given in Caltrans Bridge Design Manuals (CBDM) form the basis of design. Certain criteria herein exceed those of CBDM. For items not specifically addressed in this or other project specific Technical Memoranda, CBDM shall be used.

For OBE events, current Caltrans force based design methods and philosophies as given in Caltrans Bridge Design Specifications (CBDS) form the basis of design. Certain criteria herein exceed those of CBDS.

3.2.2 Seismic Design Philosophy

The seismic design philosophy differs depending upon the design earthquake.

3.2.2.1 MCE and FBE Design Philosophy

For MCE and FBE events, ductile structural response is required, whereby:

- The structure shall have a clearly defined and pre-determined mechanism for seismic response.
- Inelastic behavior shall be limited to columns, piers, footing foundations and abutments.
- The seismic detailing requirements per CSDC shall be satisfied.

Pre-determined structural components are allowed to have inelastic behavior. This provides a fusing mechanism, whereby the plastic response of the fuse limits the system demands. Other non-fusing components are designed as force-protected, with over-strength design providing a safe margin to resist the plastic demands.

The two main allowable fusing mechanisms for bridges and aerial structures are column flexural plastic hinging and foundation rocking.

In each case, the non-fusing or force-protected members shall be designed to prevent brittle failure mechanisms, such as footing shear, column to footing joint shear, column shear, tensile failure at the top of concrete footings, and unseating of girders. For design of force protected members, the column plastic moment and shear shall be used with over-strength (at least 120%) factors applied.

For flexural plastic hinging, it is generally desirable to limit plastic hinging to the columns. The location of plastic hinges shall be at points accessible for inspection and repair.

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 2.5. The capacity protected bridge superstructure shall remain essentially elastic.

Sacrificial components, such as abutment shear keys, are not subject to capacity protected response under MCE and FBE events. Stable rocking response is allowed for spread footing foundations.

Rocking is allowed during MCE events, as long as collapse is prevented.

Limited rocking is allowed during FBE events; they must result in small permanent offsets, not permanently interfering with functionality or serviceability.

Modeling and analysis shall conform to CBDM and CSDC.

3.2.2.2 OBE Design Philosophy

For OBE events, elastic structural response is required, whereby:

- The structure shall respond elastically under OBE response
- The track shall comply with track safety and rail-structure interaction criteria concurrent with high-speed train loading per TM 2.10.10: Track-Structure Interaction.

Rocking is not allowed for OBE events.

Verify OBE demands versus force-based capacities calculated per CBDS, with project specific amendments per Section 3.2.5.2.

3.2.2.3 Seismic Isolation

Seismic isolation may be an effective scheme to minimize damage, reduce seismic demands on substructures, and reduce foundation costs. For seismic isolation, AASHTO's Guide Specifications for Seismic Isolation Design [7] shall be used for design.

Note that seismic isolation shall contain sufficient capacity under service (i.e., braking and acceleration, wind, etc.) loads and OBE events, in order to meet criteria in TM 2.10.10: Track-Structure Interaction.

3.2.3 Seismic Demands on Structural Components

In increasing order of complexity, analysis techniques include equivalent static analysis (ESA), response spectrum analysis (RSA), equivalent linear time history analysis (ELTHA), and non-linear time history analysis (NLTHA).

The analysis technique proposed for each structure under each design earthquake shall be part of the Seismic Analysis and Design Plan.

For MCE and FBE events, the appropriate analysis technique will depend upon the site-specific conditions and complexity of the structure. The Seismic Analysis and Design Plan shall contain commentary as to the suitability of linear versus nonlinear analysis, considering geohazards, the severity of design ground motions, induced strains in the soil and structure, and expected nonlinearities

For OBE events, due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the analysis technique for the track. For the structure, an elastic analysis is appropriate.

3.2.3.1 **Force Demands (F_U) for OBE**

For OBE events, elastically calculated force demand, F_u, shall be determined for all structural components.

For the structure, the loading combination shall be as specified in TM 2.3.2: Structure Design Loads.

For the track, loading combinations for track safety and rail-structure interaction shall be as specified in TM 2.10.10: Track-Structure Interaction.

3.2.3.2 **Displacement Demands ($\triangle_D$) for MCE and FBE**

For MCE and FBE events, the displacement demand, $\triangle_D$, at the center of mass of the superstructure for each bent shall be determined, and compared versus the displacement capacity, $\triangle_C$.

For the structure, the loading combination shall be as specified in TM 2.3.2: Structure Design Loads.

3.2.3.3 **Vertical Earthquake Motions**

Vertical earthquake motions only apply to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6: Fault Rupture Analysis and Mitigation.

Structures at or in close proximity of hazardous faults shall be designed using time history analyses including consideration of horizontal and vertical earthquake motions.

3.2.3.4 **Effective Sectional Properties**

For MCE and FBE events, cracked bending and torsional moments of inertia for ductile and superstructure concrete members shall be per CSDC Section 5.6.

When moment-curvature analysis of concrete members is used, elemental cross sectional analysis shall be performed which considers the effects of concrete cracking, the degree of confinement and reinforcement yield and strain hardening, in accordance with CMTD and CSDC.

For structural steel sections, either moment-curvature analysis may be performed which consider the stress-strain relationship of the structural steel, or effective section properties presented derived based upon the degree of nonlinearity may be used. Seismic criteria for structural steel components are not presently incorporated in CSDC ver. 1.6., but will be incorporated in future releases of CSDC.

For OBE events, effective bending moments of inertia for concrete column members shall consider the maximum moment demand, M_a, and the cracking moment, M_{cr}, in accordance with CBDS Section 5.7.3.6.2. When using this method, the cracked moment of inertia, I_{cr}, shall be per CSDC Section 5.6. Alternatively, OBE effective sectional properties can be directly found through the use of moment-curvature analysis.

3.2.3.5 **Mass**

Both elemental and lumped mass may be used in analysis.

Translational and rotational elemental mass is based upon the mass density, length and cross sectional properties of discrete elements within the analysis model.

Translational and rotational lumped mass is based upon engineering evaluation of the structure, and often includes items modeled as rigid (i.e., pile and bent caps), or items not explicitly modeled (i.e., non-structural items).

3.2.3.6 **Expected Material Properties**

Expected material properties shall be used in calculating the structural seismic demands. They shall conform to CSDC for concrete members and CBDS for structural steel members.

3.2.3.7 **Flexural Plastic Hinging**

Where flexural plastic hinging is used as the primary seismic response mechanism of the structure, the analysis shall conform to CSDC methods and procedures.

3.2.3.8 Assessment of Track-Structure Interaction

For assessment of train and track-structure interaction, including requirements and load combinations which include OBE events, see TM 2.10.10: Track-Structure Interaction. For OBE events due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the appropriate analysis technique for the track. For the structure, an elastic analysis is appropriate.

3.2.3.9 Foundation Stiffness

For caissons, pile or drilled shaft foundations, the foundation stiffness shall be considered for all types of analyses. Liquefaction, lateral spreading and other seismic phenomena as specified in Section 3.2.3.14 shall be considered.

Pile foundation stiffness shall be determined through lateral and vertical pile analysis and shall consider group effects. If the foundation stiffness (translational and rotational) is large relative to the column or pier stiffness (i.e., foundation translational/rotational stiffness is 25 times greater than the column), then the foundation may be modeled as rigid.

For shallow foundations, seismic phenomena as specified in Section 3.2.6.3 shall be considered.

3.2.3.10 Boundary Conditions

In cases where the structural analysis model includes only a portion of the whole structures or abutments, the model shall also contain appropriate elements at its boundaries to capture mass and stiffness effects of the adjacent structure and/or abutment.

After completion of static or dynamic analysis, a check shall be performed to verify that the boundary conditions and element properties are consistent with initial modeling assumptions.

3.2.3.11 Continuous Welded Rail

For structures that have continuously welded rail, with either direct fixation or ballasted track, there may be benefits to the structural performance during a seismic event provided by the rail system. The rails may serve as restrainers at the expansion joists, essentially tying adjacent frames together under seismic loading. However, this is complex behavior, which must be substantiated and validated.

Since the rail system seismic response at the expansion joists is highly nonlinear, response spectrum analysis is not appropriate. Instead, a nonlinear time-history analysis in accordance with Section 3.2.3.19, shall be performed which considers rail-structure interaction.

TM 2.10.10 Track-Structure Interaction contains details of the rail-structure interaction modeling methodology. The rail-structure interaction shall include the rails and fastening system, modeled to consider fastener slippage and rail stiffness. The capacity of the fastener connections in both shear and uplift shall be accounted for in the analysis. Without these rail-structure interaction considerations, any structural performance benefits provided by continuous welded rail shall be ignored.

3.2.3.12 Train Mass and Live Load

For MCE and FBE events, trains shall not be considered.

For OBE events, train live loads with impact factor and longitudinal braking forces shall be applied to the structural system, per TM 2.3.2 Structure Design Loads, as to produce the maximum effect. The number of cars to be included in the analysis will vary depending on the adjacent span lengths. Where applicable or specific analysis methods require, CHST train loads may be modeled as equivalent static distributed loads. Where equivalent distributed loads are used in the analysis, they shall account for any local or global effects to the structure due to actual concentrated axle loads.

For single track structures, when applying loading combinations for OBE events, the following train effects shall be considered simultaneously:

1. One train vertical live load + impact
2. One train longitudinal braking force

3. Mass of one train, applied at the center of mass of the train

For multiple track structures, ½ of trains potentially occupying the structure shall be considered. Where an odd number of trains potentially occupy the structure, round down to the nearest whole number of trains (example: for 3 trains, use ½(3) = 1.5 → round down to 1). When applying load combinations for OBE events, the following train effects shall be considered simultaneously:

1. ½ of the trains live load + impact
2. ½ of trains longitudinal braking force
3. Mass of ½ of the trains, applied at the center of mass of the trains

For structural design, the OBE loading combination shall be as specified in TM 2.3.2 Structure Design Loads.

For the track and when considering track-structure interaction, OBE loading combinations for track safety and rail-structure interaction shall be as specified in TM 2.10.10 Track-Structure Interaction.

3.2.3.13 P-△ Effects

For flexural plastic hinging, P-△ effects shall conform to the requirements in CSDC.

3.2.3.14 Soil Structure Interaction

For soil-structure interaction (SSI) modeling and analysis procedures, see TM 2.9.10 Geotechnical Design Guidelines.

3.2.3.15 Displacement Demand Amplification Factor

When equivalent static analysis (ESA) or response spectrum analysis (RSA) is used for MCE or FBE events, the displacement demand, $\triangle_D$, obtained shall be multiplied by an amplification factor, C, as follows:

For $T_i/T_o < 1$: $C = [0.8/ (T/T_o)] + 0.2$

For $T_i/T_o > 1$: $C = 1.0$

where:

T_i = fundamental period of structure in the longitudinal or transverse direction (including foundation stiffness)

T_o = the period centered on the peak of the longitudinal or transverse acceleration response spectrum

In order to account for the uncertainty associated with calculation of structural period for stiff structures.

3.2.3.16 Equivalent Static Analysis

Equivalent static analysis (ESA) may be used to determine earthquake demands, E:

- For MCE and FBE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure.
- For OBE events, the Force Demands, F_u

when the structure can be characterized as a simple single-degree-of-freedom (SDOF) system, and dynamic analysis will not add significantly more insight into behavior.

ESA shall apply to standard or non-standard bridge or aerial structures having no skew, and having single column piers or multiple column bents where most of the structural mass is concentrated at a single level. ESA is applicable for bridges, aerial structures, or individual frames with the following characteristics:

- Response primarily captured by the fundamental mode of vibration with uniform translation.

- Simply defined lateral force distribution (e.g. balanced spans, approximately equal bent stiffness)
- No skew

ESA shall not apply to complex bridge or aerial structures as defined in Section 2.6.3.

ESA earthquake demands shall be determined from horizontal spectra by either of two methods:

- Method 1: Earthquake demand, $E = (E_L^2 + E_T^2)^{1/2}$, where E_L and E_T are the responses due to longitudinal and transverse direction earthquake motions as defined below. The application of ground motion shall be along the principal axes of individual components.
- Method 2: Earthquake demand, E, by using the 100%-30% rule, for two cases::

  Case 1 : $E = 1.0E_L + 0.3E_T$

  Case 2 : $E = 0.3E_L + 1.0E_T$

For calculation of ESA earthquake demands:

Longitudinally: $E_L = C * S_a^L * W$

Transversely: $E_T = C * S_a^T * W$

Where:

C = the amplification factor, C, given in Section 3.2.3.15,

S_a^L = longitudinal acceleration response spectral value at period T_L.

T_L = fundamental period of structure in the longitudinal direction (including foundation stiffness)

S_a^T = transverse acceleration response spectral value at period T_T.

T_T = fundamental period of structure in the transverse direction (including foundation stiffness)

W = tributary dead load + superimposed dead load for MCE and FBE

W = tributary dead load + superimposed dead load + live load for OBE per Section 3.2.3.12.

Effective sectional properties shall be used per Section 3.2.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE and FBE events, 5% damped response spectra shall be used to determine S_a.

For OBE events, 3% damped response spectra shall be used to determine S_a.

3.2.3.17 **Response Spectrum Analysis**

Response spectrum analysis (RSA) shall be used to determine earthquake demands, E:

- For MCE and FBE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when ESA does not provide an adequate estimate of the dynamic behavior.

RSA shall apply to standard or non-standard bridge or aerial structures having skewed bents or abutments ≤ 45°, and having single column piers or multiple column bents. RSA is applicable for bridges or aerial structures with the following characteristics:

- Response primarily captured by the fundamental structural mode shapes containing a minimum of 90% mass participation in the longitudinal and transverse directions.

- Skewed bents or abutments ≤ 45°,

RSA shall not apply to complex bridge or aerial structures as defined in Section 2.6.3.

RSA involves creating a linear, three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics. The dynamic model is used to determine the fundamental structural mode shapes for use in analysis. A sufficient number of modes shall be included to account for a minimum of 90% mass participation in the longitudinal and transverse directions. Care shall be taken to ensure 90% mass participation for long viaduct models. The designer shall examine the modes to ensure that they sufficiently capture the behavior of the structure.

A linear elastic multi-modal spectral analysis shall be performed using the appropriately damped response spectra, as given in the Geotechnical Data Report. The modal response contributions shall be combined using the complete quadratic combination (CQC) method.

For MCE and FBE events, RSA based on design spectral accelerations will likely predict forces in some elements that exceed their elastic limit, the presence of which indicates nonlinear behavior. The designer shall recognize that forces generated by RSA could vary considerably from the actual force demands on the structure. Sources of nonlinear response not captured by RSA include the effects of surrounding soil, yielding of structural members, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior.

Where there is a change in soil type along the bridge alignment or the bridge is complex, consideration shall be made to the possibility that out-of-phase ground displacements at two adjacent piers may increase the computed demand on expansion joints, rails or columns. This effect is not explicitly considered in RSA. In such cases, more sophisticated time history analyses shall be used.

Appropriate linear stiffness shall be assumed for abutments and expansion joints. Analyses shall be performed for compression models (abutments engaged, gaps between frames closed) and for tension models (abutments inactive, gaps between frames open), to obtain a maximum response envelope. If analysis results show that soil capacities are exceeded at an abutment, iterations shall be performed with decreasing soil spring constants at the abutment per CBDS and CMTD recommendations.

For calculation of differential displacements at expansion joints and for calculation of column drift, the analysis shall either explicitly compute these demands as modal scalar values or assume that the displacements and rotations combine to produce the highest or most severe demand on the structure.

RSA earthquake demands shall be determined from horizontal spectra by either of two methods:

- <u>Method 1:</u> Earthquake demand, $E = (E_L^2 + E_T^2)^{1/2}$, where E_L and E_T are the responses due to longitudinal and transverse earthquake spectra as defined below. The application of ground motion shall be along the principal axes of individual components.
- <u>Method 2:</u> Earthquake demand, E, by using the 100%-30% rule, for two cases::

Case 1 : $E = 1.0E_L + 0.3E_T$

Case 2 : $E = 0.3E_L + 1.0E_T$

For calculation of RSA earthquake demands:

Longitudinally: E_L = C * (RSA demands from longitudinal earthquake spectra)

Transversely: E_T = C * (RSA demands from transverse earthquake spectra)

Where:

C = the amplification factor, C, given in Section 3.2.3.15,

Effective sectional properties shall be used per Section 3.2.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE and FBE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 3.2.3.12.

For MCE and FBE events, 5% damped response spectra shall be used.

For OBE events, 3% damped response spectra shall be used.

3.2.3.18 Equivalent Linear Time History Analysis

Equivalent linear time history analysis (ELTHA) shall be used to determine earthquake demands, E:

- For MCE and FBE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when ESA or RSA provides an unrealistic estimate of the dynamic behavior.

ELTHA shall apply to standard or non-standard bridge or aerial structures having skewed bents or abutments > 45°, since the directionality of seismic motions for highly skewed structures is an important consideration.

ELTHA shall not apply to complex bridge or aerial structures as defined in Section 2.6.3.

ELTHA involves creating a three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics.

For MCE and FBE events, motions consistent with the 5% damped response spectra shall be used.

For OBE events, motions consistent with the 3% damped response spectra shall be used.

Rayleigh damping shall be used for ELTHA. The form of damping requires the calculation of both stiffness and mass proportional coefficients anchored at two structural frequencies, which shall envelope all important modes of structural response. The lowest structural frequency (i.e., longest period) shall be one anchor frequency, the other shall be chosen such that a minimum of 90% mass participation in the longitudinal and transverse directions are enveloped. To determine the frequency anchor at the low structural frequency, the frequency analysis shall be performed using cracked section properties and the resulting frequency reduced by 10%.

For MCE and FBE events, Rayleigh damping shall be 5%.

For OBE events, Rayleigh damping shall be 3%.

Effective sectional properties shall be used per Section 3.2.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE and FBE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 3.2.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling design earthquake ground motions, as given in the Geotechnical Data Report. The motions shall consist of two-horizontal ground motion time histories, selected, scaled, and spectrally matched. The two horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motions relative to the local or global coordinate systems of the structural model.

Vertical earthquake time histories shall also be applied to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6: Fault Rupture Analysis and Mitigation. In such cases, the motions shall consist of two horizontal and one vertical ground motion time histories, selected, scaled, and spectrally matched.

When ELTHA is used, the following analyses shall be performed:

- Seven sets of ground motions, the average value of each response parameter (e.g.: force or strain in a member, displacement or rotation at a particular location) shall be used for design.

After completion of each ELTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

3.2.3.19 **Nonlinear Time History Analysis**

Nonlinear time history analysis (NLTHA) shall be used to determine earthquake demands, E:

- For MCE and FBE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when RSA or ELTHA provides an unrealistic estimate of the dynamic behavior, provides overly conservative demands, or where nonlinear response is critical for design.

NLTHA shall apply to complex bridge or aerial structures.

For OBE events, due to track-structure interaction requirements (per TM 2.10.10: Track-Structure Interaction) which require nonlinear fastener slippage, NLTHA shall be the analysis technique for the track, regardless of the structural classification. For the structure, ESA, RSA, or ELTHA analysis may be appropriate, dependent upon the requirements for each analysis above.

NLTHA involves creating a three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics. This dynamic model is used to determine the dynamic characteristics of the structure by including selected nonlinear representations of structural and foundation elements.

For MCE and FBE events, motions consistent with the 5% damped response spectra shall be used.

For OBE events, motions consistent with the 3% damped response spectra shall be used.

Rayleigh damping shall be used for NLTHA. The form of damping requires the calculation of both stiffness and mass proportional coefficients anchored at two structural frequencies, which shall envelope all important modes of structural response. The lowest structural frequency (i.e., longest period) shall be one anchor frequency, the other shall be chosen such that a minimum of 90% mass participation in the longitudinal and transverse directions are enveloped. To determine the frequency anchor at the low structural frequency, the frequency analysis shall be performed using cracked section properties and the resulting frequency reduced by 10%.

For MCE and FBE events, Rayleigh damping shall be 5%.

For OBE events, Rayleigh damping shall be 3%.

Where applicable, effective sectional properties shall be used per Section 3.2.3.4. Otherwise, cross sectional properties of concrete and steel elements with nonlinear behavior may be represented by moment-curvature relations.

Where applicable, equivalent linear representation of foundation stiffness shall be used, and iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response). Otherwise, nonlinear representations of foundation characteristics shall be used.

For MCE and FBE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 3.2.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling design earthquake ground motions, as given in the Geotechnical Data Report. The motions shall consist of two horizontal ground motion time histories, selected, scaled, and spectrally matched. The two horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motion relative to the local or global coordinate systems of the structural model.

Vertical earthquake time histories shall also be applied to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6: Fault Rupture Analysis and Mitigation. In such cases, the motions shall consist of two horizontal and one vertical ground motion time histories, selected, scaled, and spectrally matched. When NLTHA is used, the following analyses shall be performed:

- Seven sets of ground motions, the average value of each response parameter (e.g.: force or strain in a member, displacement or rotation at a particular location) shall be used for design.

After completion of each NLTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

3.2.3.20 Rocking for MCE and FBE

For MCE and FBE events, where rocking of the footings is used as the primary seismic response mechanism of the structure, nonlinear analysis methods are required. One acceptable method for such analysis is the most current Caltrans rocking analysis procedure, which includes the following steps:

1. Develop a relationship between the top of the column displacement and the rocking period of the footing.
2. Develop a displacement response spectrum from the design acceleration response spectrum or use the displacement response spectrum provided in the design criteria (note: the designer shall account for greater damping associated with rocking behavior as recommended in the Caltrans procedure.).
3. Begin with an initial assumed total displacement. Use a computational approach that produces a calculated total displacement.
4. If the calculated displacement equals the initial assumed displacement, convergence is reached and a stable rocking response found.
5. If the calculated displacement differs from the initial assumed displacement, then convergence not is reached. Resize the footing and iterate until convergence is reached.

When determining the rocking response of an aerial structure, consideration shall be given to possible future conditions, such as a change in depth of the soil cover above the footing or other loads that may increase or decrease the rocking response.

An alternative to the method described above, a more rigorous analysis of the rocking response shall be performed using a NLTHA.

3.2.4 Seismic Capacities of Structural Components

3.2.4.1 **Force Capacities (ΦF_N) for OBE**

For OBE design, LRFD force capacities, $\Phi F_{N,}$ for all structural components shall be found in accordance with CBDS.

3.2.4.2 **Displacement Capacity ($\triangle_C$) for MCE and FBE**

For MCE and FBE design using ESA, RSA, and ELTHA demands, the displacement capacity, $\triangle_C$, shall be determined by nonlinear static displacement capacity or "pushover analysis" as described in Section 3.2.4.3. 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. Specified capacity shall be considered to be reached when the concrete or steel strains of any primary element meets the limits specified in Sections 3.2.4.5 to 3.2.4.8.

For comparison to NLTHA demands, if a moment curvature representation of plastic hinging is used, then the curvature demands shall be converted to concrete or steel strains, and verified versus allowable strains in Sections 3.2.4.5 to 3.2.4.8.

The displacement capacity, $\triangle_C$, shall include all displacements attributed to flexibility in the foundations, bent caps, and other elastic and inelastic member responses in the system. The assumptions made to determine the displacement capacity, $\triangle_C$, shall be consistent with those used to determine the displacement demand, $\triangle_D$.

All structural members and connections shall also satisfy the capacity based performance requirements in Section 3.2.6.

3.2.4.3 **Nonlinear Static Analysis**

For MCE and FBE events, in determining the displacement capacity, $\triangle_C$, using nonlinear static pushover analysis the following procedure shall be followed:

Dead load shall be applied as an initial step.

Incremental lateral displacements shall be applied to the system. A plastic hinge shall be assumed to form in an element when the internal moment reaches the idealized yield limit in accordance with Section 3.2.3.7. The sequence of plastic hinging through the frame system shall be tracked until an ultimate failure mode is reached. The system capacity shall then be determined in accordance with CSDC.

3.2.4.4 **Plastic Hinge Rotational Capacity**

Plastic moment capacity of ductile flexural members shall be calculated by moment-curvature (M-φ) analysis and shall conform to CSDC for concrete members and CBDS for structural steel members.

The rotational capacity of any plastic hinge is defined based on the curvature in M-φ analysis where the structural element first reaches the allowable strain limits described below.

3.2.4.5 **Strain Limits for Ductile Reinforced Concrete Members**

For MCE and FBE events, the following reinforcing steel (A706/Grade 60) allowable tensile strain limits ($\varepsilon_{su}{}^{a}$) shall apply for ductile reinforced concrete members:

MCE: $\varepsilon_{su}{}^{a} \leq 2/3\ \varepsilon_{su}$

FBE: $\varepsilon_{su}{}^{a} \leq \varepsilon_{sh}$

where: ε_{su} = ultimate tensile strain per CSDC

ε_{sh} = tensile strain at the onset of strain hardening per CSDC

For MCE and FBE events, the following allowable confined concrete compressive strain limits $\varepsilon_{cu}{}^{a}$) shall apply for ductile reinforced concrete members:

MCE: $\varepsilon_{cu}{}^{a} \leq 2/3\ \varepsilon_{cu}$

FBE: $\varepsilon_{cu}^{a} \le$ lesser of 1/3 ε_{cu} or 1.5 ε_{cc}

where: ε_{cu} = ultimate compressive strain as computed by Mander's model for confined concrete.

ε_{cc} = strain at maximum concrete compressive stress as computed by Mander's model for confined concrete.

3.2.4.6 Strain Limits for Ductile Reinforced Concrete Caissons, Piles, and Drilled Shafts

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 2.5.

For MCE events, the following reinforcing steel (A706/Grade 60) allowable tensile strain limit ε_{su}^{a}) shall apply for ductile reinforced concrete caissons, piles, and drilled shafts:

MCE: $\varepsilon_{su}^{a} \le \varepsilon_{sh}$

where: ε_{sh} = tensile strain at the onset of strain hardening per CSDC

For MCE events, the following allowable confined concrete compressive strain limits (ε_{cu}^{a}) shall apply for ductile reinforced concrete caissons, piles, and drilled shafts:

MCE: $\varepsilon_{cu}^{a} \le$ lesser of 1/3 ε_{cu} or 1.5 ε_{cc}

where: ε_{cu} = ultimate compressive strain as computed by Mander's model for confined concrete.

ε_{cc} = strain at maximum concrete compressive stress as computed by Mander's model for confined concrete.

3.2.4.7 Strain Limits for Unconfined Concrete

Unconfined compressive strain limits shall 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 shall be considered unconfined.

For MCE and FBE events, the following allowable concrete unconfined compressive strain limits ε_{cu}^{a}) apply:

MCE: $\varepsilon_{cu}^{a} = 0.004$

FBE: $\varepsilon_{cu}^{a} = 0.0035$

There are no allowable strain requirements for unconfined cover concrete.

3.2.4.8 Strain Limits for Structural Steel Elements

For MCE and FBE events, the following structural steel allowable tensile strain limits (ε_{su}^{a}) apply:
MCE: $\varepsilon_{su}^{a} \le 2/3\ \varepsilon_{su}$
FBE: $\varepsilon_{su}^{a} \le \varepsilon_{sh}$
where: ε_{su} = ultimate tensile strain

ε_{sh} = strain at the onset of strain hardening

Structural steel allowable compressive strain limits shall be determined based upon governing local or global buckling in accordance with CBDS, using expected material properties.

3.2.4.9 Rocking

The rocking capacity of the bridge and aerial structure piers shall be determined as per Section 3.2.3.20.

3.2.4.10 Expected Material Properties

Expected material properties shall be used in calculating structural seismic capacities, except shear. For seismic shear capacities, use nominal material properties. Expected material properties shall conform to CSDC for concrete members and CBDS for structural steel members.

3.2.4.11 **Shear Capacity**

Shear capacity of ductile components shall conform to CSDC for concrete members and CBDS for structural steel members.

3.2.4.12 **Joint Internal Forces**

For all events, continuous force transfer through the column/superstructure and column/footing joints shall conform to CSDC. These joint forces require that the joint have sufficient strength to ensure elastic behavior in the joint regions based on the capacity of the adjacent members.

3.2.5 Seismic Performance Evaluation

3.2.5.1 **Rocking**

For MCE and FBE events, when rocking is the primary seismic response mechanism, a stable rocking response must be provided, see Section 3.2.3.20.

For OBE events, rocking of structures is not allowed.

3.2.5.2 **Force Based Design for OBE**

For OBE events, the maximum force based Demand/Capacity Ratio shall be:

$$F_U / \Phi F_N \leq 1.0$$

Where:

F_U = the force demand, as defined in Section 3.2.3.1.

ΦF_N = the LRFD force capacity, as defined in Section 3.2.4.1.

in order to satisfy the OPL performance objectives specified in Section 2.7.2. See TM 2.3.2 Structure Loads for applicable load combinations.

3.2.5.3 **Displacement Based Design for MCE and FBE**

For MCE and FBE events, the maximum displacement Demand/Capacity Ratio shall be:

$$\triangle_D / \triangle_C \leq 1.0$$

Where:

$\triangle_D$ = the displacement demand, as defined in Section 3.2.3.2.

$\triangle_C$ = the displacement capacity, based on strain limits, as defined in Section 3.2.4.2.

in order to satisfy the NCL and FPL performance objectives specified in Section 2.7.2.

3.2.5.4 **Demand versus Capacity Evaluation**

Demand/capacity ratios in any three orthogonal directions may be evaluated separately for columns and footings.

For other members which carry vertical loads primarily through bending, such as superstructure members and bent caps, vertical dead and seismic D/C ratios shall be evaluated in combination with the horizontal seismic D/C ratios. In evaluating the combined D/C ratios, 1.0, 0.3, 0.3 rules shall be used for the seismic loads. The vertical dead load shall always have a factor of 1.0 applied.

When evaluating seismic loads on piles or drilled shafts, vertical and horizontal seismic loads need not be combined. However, the designer shall evaluate the piles with the column plastic moment acting about the principal axes, as well as about diagonal axes to determine the critical loading on the piles.

3.2.6 Seismic Design

All structure design shall conform to the requirements specified herein and CBDM.

3.2.6.1 **Capacity Protected Element Design**

In order to limit the inelastic deformations to the 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 (at least 120%) design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC for concrete members and CBDS for structural steel members.

3.2.6.2 **Soil Improvement**

For details of soil improvement design, see TM 2.9.10: Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding soil improvement.

3.2.6.3 **Design of Shallow Foundations**

For details of shallow foundation design, see TM 2.9.10: Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding design of shallow foundations.

Shallow foundations shall be designed as capacity protected structural elements under any loading or combination of loadings, including seismic loads. When designing for footing shear, column-to-footing joint shear, and moments in footings, the column plastic moment and shear shall be used with 120% overstrength factors applied.

Under OBE events, foundation rocking shall not be allowed and the soil pressure diagram shall have a compressive width of at least half of the footing width.

3.2.6.4 **Design of Caisson, Pile, and Drilled Shaft Foundations**

For details of caisson, pile, and drilled shaft foundation design, see TM 2.9.10: Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding these types of foundations, such as:

- Ultimate and design load capacities in compression and tension
- Negative skin friction or down drag forces
- Resistance to lateral loads
- Group effects
- Allowable differential settlements
- Battered piles

Caisson, pile and drilled shaft foundations shall be designed as capacity protected structural elements under any loading or combination of loadings, including seismic loads. When designing for pile/drilled shaft cap shear, column-to-pile/drilled shaft cap joint shear, and moments in pile/drilled shaft cap, the column plastic moment and shear shall be used with 120% over strength factors applied.

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 2.5.

The design of piles shall be in accordance with the CBDM. The CBC special detailing requirements for seismic Zones 3 and 4 shall also be applicable to the pile design for bridges and aerial structures.

Full corrosion protection shall be provided for steel piles in the form of cathodic protection or through a corrosion allowance added to the steel section thickness.

3.2.6.5 **Battered Piles**

The use of battered piles shall, to all practical extents, be avoided. Where the use of battered piles is unavoidable, due to their relative stiffness they must carry all of the expected lateral demands, since in such scenarios vertical piles provide little lateral resistance. Where battered piles are used, displacement-strength compatibility must be considered.

Battered piles shall be designed to safely resist all imposed loadings, including resistance to crushing at the pile-pile cap interface under seismic loading. In addition, development of the pile reinforcing into the pile cap shall consider the additional significant tensile demands on these piles and potential shear failure of the piles under concurrent tensile demands. Battered piles shall not be allowed where negative skin friction is anticipated.

Battered piles shall not be farther out of plumb than one horizontal unit in three vertical units.

Where battered piles are to be used, consideration shall be given to the possibility of such battered piles encroaching on property outside the right-of-way, or interfering with existing structures or pile foundations.

3.2.6.6 Expansion Joint and Hinge / Seat Capacity

The detailed design of structural expansion joints shall provide free movement space for creep, shrinkage, temperature variation, braking and acceleration, and seismic response.

Under MCE and FBE response, structural expansion joints shall be verified to ensure that damaged joints will not induce changes to important structural behavior. Only local damage is acceptable.

Adequate seat length shall be provided to accommodate anticipated seismic displacements and prevent unseating of the structure. Seat width requirements are specified in CSDC for hinges and abutments. Hinge restrainers shall be designed as a secondary line of defense against unseating of girders in accordance with CSDC.

When excessive seismic displacement must be prevented, shear keys shall be provided and designed as capacity-protected elements.

Transverse shear keys shall be provided to accommodate the anticipated seismic loads without modification to the provision for thermal movement and vibration characteristics.

3.2.6.7 Columns

Columns shall satisfy the detailing requirements for ductile structural elements as specified in CSDC.

3.2.6.8 Superstructures

Superstructures shall be designed as capacity protected elements, and shall remain essentially elastic.

3.2.6.9 Structural Joints

Superstructure and the bent cap joints and footing joints shall conform to the requirements of CSDC.

3.3 TUNNELS AND UNDERGROUND STRUCTURES

3.3.1 General

Bored tunnels, cut-and-cover tunnels, mined tunnels, portals, U-sections, ventilation structures, and other underground structures, which directly support high-speed train service, are Primary Structures.

For seismic design criteria for earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10: Geotechnical Design Guidelines.

This document does not discuss culverts, pipelines or sewer lines, nor does it specifically discuss issues related to deep chambers such as hydropower plants, mine chambers, and protective structures. Future Technical Memoranda for those items are pending.

3.3.2 Design Codes

Generally, current Caltrans seismic analysis and design philosophies as stated in Caltrans Bridge Design Manuals (CBDM) form the basis of design. However, certain criteria herein exceed those of CBDM. For items not specifically addressed in this or other project specific Technical Memoranda, CBDM shall be used.

3.3.3 Seismic Design Philosophy

For tunnels and underground structures, the intended structural action under seismic loading is that of a Ductile Structure, whereby:

- The tunnel or underground structure shall have a clearly defined mechanism for response to seismic loads.
- Inelastic behavior shall be limited to selected regions, the remainder of the structure shall be force protected to prevent brittle failure mechanisms.

In general, the designer allows specified structural components to undergo inelastic behavior under MCE and FBE events, while force-protecting other components. The structure shall remain elastic under the OBE events.

An adequate margin of strength shall be provided between the designated load-resistance ductile mode and non-ductile failure modes. Sufficient over-strength capacity (at least 120%) shall be provided to assure the desired ductile mechanism occurs and that the undesirable non-ductile failure mechanisms are prevented from forming.

3.3.4 Seismic Demands on Structural Components

3.3.4.1 General

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: racking/ovaling, axial, and curvature deformations.

1. Racking/ovaling deformations primarily due to seismic waves propagating transverse to the tunnel axis.
2. Axial deformations primarily due to seismic waves along the tunnel axis.
3. Curvature deformations primarily due to seismic waves along the tunnel axis.

Appropriate modeling and analysis methods shall be used for static and seismic analyses of the tunnels and portal structures.

3.3.4.2 Input Ground Displacements and Velocities

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.

Ground displacements and velocities are primary considerations for the seismic design of underground structures. To assess the ground displacements and velocities induced by the design earthquakes, the effects of soil nonlinearity and soil-structure interaction shall be considered. Special problems related to the site, such as liquefaction, fault rupture and excessive settlement, shall be evaluated and taken into consideration per the Geotechnical Data Report.

Ground displacements shall be in accordance with TM 2.9.6: Interim Ground Motion Guidelines.

Soil springs, both laterally (p-y) and vertically (t-z), shall be in accordance with the Geotechnical Data Report.

For shallow buried structures in close proximity (R<20 km) to hazardous earthquake faults where seismic loadings may produce a significant inertia response, vertical effects must be considered. In such cases, the dynamic motions applied shall consist of two horizontal and one vertical ground motion time-histories, selected, scaled and spectrally matched.

The time-history analysis should include: Seven sets of ground motions, the average value of each response parameter (e.g.: force or strain in a member, displacement or rotation at a particular location) shall be used for design. After completion of each NLTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

3.3.4.3 **Analysis Techniques**

The general procedure for seismic design of underground structures shall be based primarily on the ground deformation approach. During earthquakes, underground structures move together with the surrounding geologic media. The structures, therefore, shall be designed to accommodate the deformations imposed by the ground. The relative stiffness between the underground structure and surrounding soil shall be considered; the effects of soil-structure interaction shall be taken into consideration.

3.3.4.4 **Load and Load Combinations**

The seismic design and evaluation of tunnels and underground structures shall consider loading and load combinations as given in TM 2.3.2: Structure Design Loads.

3.3.4.5 **Construction Sequence**

Construction sequence including dead loads, surcharge, and potential soil arching effects shall be included as initial conditions, occurring prior to the seismic demands.

3.3.4.6 **Capacity Reduction Factors**

For evaluating the capacity protected seismic response of underground tunnels, capacity reduction factors in accordance with CBDM shall be used.

3.3.4.7 **Proximity Analysis**

If a tunnel is built in the vicinity of another tunnel, underground structure, or at-grade structure, a proximity study shall be performed. The results, conclusions, and subsequent analysis requirements of the proximity study shall be submitted to the Authority or delegate for review and comment.

3.3.4.8 **Racking/Ovaling Analysis**

Racking/ovaling deformations are primarily due to seismic waves propagating transverse to the tunnel axis. The deformations and strains due to these motions, which result in tunnel cross-sectional distortion, shall be evaluated by numerical methods.

As verification to numerical results, closed-form approximations of racking/ovaling demands can be found based upon the procedures outlined in [4, 5, 6, 9, 10].

3.3.4.9 **Seismic Loads due to Axial and Curvature Deformations**

Axial and curvature deformations are primarily due to seismic waves along the tunnel axis.

A global three-dimensional model of the tunnel shall be developed using either linear or nonlinear beam elements, as appropriate, representing the cross section of the tunnel.

The tunnel model shall be supported by either linear or nonlinear soil springs in the three orthogonal directions, as specified in the Geotechnical Data Report.

The ground motions, in accordance with TM 2.9.6: Interim Ground Motion Guidelines, shall be applied to the ground nodes of the springs.

3.3.4.10 **Cross Passages and Connection Joints**

The effects of stress concentration at cross passage and connection joints to the main tunnel shall be obtained using detailed three-dimensional tunnel/soil models.

3.3.4.11 **Stability**

When segmental linings are used for a bored tunnel, the stability of the segments shall be verified by the use of detailed finite element models using nonlinear soil continuum and proper contact surfaces at the segment interfaces. Racking/ovaling analysis shall be performed to examine the separation of the segments and stability of the entire system.

3.3.4.12 **Interface Joints**

Interfaces between bored tunnel structures and the more massive structures, such as the cut-and-cover structures, stations, and ventilation/access structures, shall be designed and detailed as flexible joints to accommodate the differential movements. The design differential movements shall be determined by the designer in consultation with the Geotechnical Engineer.

3.3.5 Seismic Capacities of Structural Components

3.3.5.1 Earth Embankments, Retaining Structures

For seismic design criteria for earth supporting structures, such as earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10: Geotechnical Design Guidelines.

Information contained within the Geotechnical Data Report shall form the basis of design.

3.3.5.2 Cut-and-Cover Tunnels

For seismic design of cut-and-cover tunnels, CBDM and additional requirements in Geotechnical Data Report form the basis of design.

3.3.5.3 Tunnel Portals

Seismic design criteria for tunnel portals are under final development and approval.

Where tunnel portals consist of reinforced concrete structures, then CBDM shall form the basis of design.

3.3.5.4 Bored Tunnels

Bored tunnels include earth tunnel sections and rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining.

Seismic design criteria for bored tunnels are under final development and approval.

Where bored tunnels have reinforced concrete lining, then CBDM shall form the basis of design.

Bored tunnel sections shall be designed to sustain all the loads to which they will be subjected to, such as:

- Handling loads as determined by the transport and handling system.
- Shield thrust ram loads as determined by the shield propulsion system.
- Erection loads including external grouting loads.
- Vertical and horizontal earth pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.
- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it
- Surcharge loads due to adjacent buildings.
- Seismic demands as indicated in this TM.

Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area.

Provisions for soil-structure interaction and lateral support of surrounding ground shall be included.

3.3.5.5 Mined Tunnels

Mined tunnels include rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining.

Seismic design criteria for mined tunnels are under final development and approval.

Where mined tunnels have reinforced concrete lining, then CBDM shall form the basis of design.

Temporary Support Systems

Temporary support systems shall be designed to sustain all the loads to which they will be subjected, such as:

- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Additional loads due to the driving of adjacent tunnels.
- Surcharge loads due to adjacent buildings.

Cast-in-Place Liners

Cast-in-place liners shall be designed to sustain all the loads to which they will be subjected, such as:

- Handling loads as determined by the transport and handling system.
- Erection loads including external grouting loads.
- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it
- Surcharge loads due to adjacent buildings.
- Seismic demands as indicated in this TM.

Precast Segmental Liners

The precast segmental liners shall be designed to sustain all the loads to which they will be subjected, such as:

- Handling loads as determined by the transport and handling system.
- Shield thrust ram loads if applicable as determined by the shield propulsion system.
- Erection loads including external grouting loads.
- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.
- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it.
- Surcharge loads due to adjacent buildings.

- Seismic demands as indicated in this TM.

Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area.

Provisions for soil-structure interaction and lateral support of surrounding ground shall be included.

3.3.5.6 **Ventilation and Access Shafts**

Seismic design criteria for ventilation and access shafts are under final development and approval.

Where ventilation and access shafts have reinforced concrete lining, then CBDM shall form the basis of design.

The seismic considerations for the design of vertical shaft structures are similar to those for bored tunnels, except that racking/ovaling and axial deformations in general do not govern the design.

Consideration shall be given to the curvature strains and shear forces of the lining resulting from vertically propagating shear waves. Force and deformation demands may be considerable in cases where shafts are embedded in deep, soft soils. In addition, potential stress concentrations at the following critical locations along the shaft shall be properly assessed and designed for: (1) abrupt change of the stiffness between two adjoining geologic layers, (2) shaft/tunnel or shaft/station interfaces, and (3) shaft/surface building interfaces. Flexible connections shall be used between any two structures with different stiffness and mass in poor ground conditions.

3.4 PASSENGER STATIONS AND BUILDING STRUCTURES

3.4.1 General

All at-grade, elevated or underground passenger stations and building structures supporting high-speed train service are categorized as Primary Structures.

3.4.2 Design Codes

CBC methodology shall be used for all non-seismic related design. However, since the CBC primarily uses force-based seismic design, ASCE 41 is referenced for the performance (i.e., strain and deformation) based seismic design methodology proposed for the CHSTP.

Although ASCE 41 is a document originally issued for seismic rehabilitation of existing structures, it is pertinent here since it is very thorough and comprehensive. It is referenced in absence, at this date, of a similar performance based code for the seismic design of new building structures.

ASCE 41 is to be used to satisfy the no collapse performance level (NCL) during the Maximum Considered Earthquake (MCE).

Although the basis of the following criteria relies heavily on ASCE 41, certain criteria might exceed those of ASCE 41. If items are not specifically addressed in this or any other section of the criteria, ASCE 41 is to be used.

Passenger stations or building structures supporting high-speed train service shall withstand the effects of the Operating Basis Earthquake (OBE) within structural deformations as given in TM 2.10.10: Track-Structure Interaction, in order to limit rail stresses and protect against derailment.

3.4.3 Seismic Design Philosophy

The intended structural action under seismic loading is:

- A “weak beam strong column” philosophy shall be implemented in the design of the buildings. The plastic hinges shall form in the beams and not in the columns. Proper detailing shall be implemented to avoid any kind of nonlinearity or failure in the joints, either ductile or brittle. The formation of a plastic hinge shall take place in the beam element at not less than twice the depth of the beam away from the face of the joint by adequate detailing.

- The building shall have a clearly defined mechanism for response to seismic loads with clearly defined load path and load carrying systems.
- Each component shall be classified as primary or secondary, and each action shall be classified as deformation-controlled (ductile) or force-controlled (nonductile). The building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application to the final point of resistance. All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level.
- The detailing and proportioning requirements for full-ductility structures shall be satisfied. No brittle failure shall be allowed.

In general, the designer may allow specified structural components to undergo inelastic behavior under the MCE and FBE, while force-protecting other components. The main nonlinear mechanism is member flexural plastic hinging. The force-protected members shall be designed to prevent brittle failure mechanisms.

The structure shall remain elastic under the OBE. Active, semi-active and passive energy dissipation devices or base isolation systems are permitted. If employed, these devices and systems are a source of nonlinear mechanism in the structure, and nonlinear analysis shall be performed.

An adequate margin of strength shall be provided for nonlinear elements. Over-strength (no less than 120%) shall be provided to assure the desired nonlinear behavior and that the undesirable non-ductile failure mechanisms are prevented from forming. All structural components not pre-determined for rocking or flexural plastic hinging shall be designed to remain essentially elastic under seismic loads. Structural components can be considered essentially elastic when the induced strains exceed elastic limits, but the resulting structural damage is minor and will not reduce the ability of the structure to carry operational loads in the near and long term. For design of force protected members, the column plastic moment and shear shall be used with the appropriate over-strength factors (at least 120%) applied.

3.4.4 Seismic Demands on Structural Components

3.4.4.1 Analysis Techniques - General

The station or building shall be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Soil-structure interaction shall be considered in the modeling and analysis, where necessary.

Structures shall be analyzed using Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP) or Nonlinear Dynamic Procedure (NDP).

Unless it is shown that the conditions and requirements for Linear Dynamic Procedure (LDP) or Nonlinear Static Procedure (NSP) can be satisfied, all structures shall be analyzed using Nonlinear Dynamic Procedure (NDP).

3.4.4.2 Linear Dynamic Procedure (LDP)

Linear dynamic procedure (LDP) shall be used in accordance with the requirements of ASCE 41. This can be either a response spectrum method or time-history method as applicable. Buildings shall be modeled with linear elastic stiffness and equivalent viscous damping values consistent with the behavior of the components responding at or near yield level, as defined in ASCE 41.

When response spectrum analysis is used, modal combination shall be performed using the CQC approach, while spatial combination shall be performed using the SRSS technique.

When LDP is used, the analysis shall be performed under seven sets of ground motions, the average value of each response parameter (e.g.: force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 2.7.3.

For buildings that have one or more of the following conditions, linear dynamic procedures (LDP) shall <u>not</u> be used:

- In-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.1 of ASCE 41.
- Out-of-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.2 of ASCE 41.
- Weak Story Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.3 of ASCE 41.
- Torsional Strength Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.4 of ASCE 41.
- Building structures subject to potential foundation sliding, uplift and/or separation from supporting soil (near field soil nonlinearity).
- Building structures which include components with nonlinear behavior such as, but not limited to, buckling, expansion joint closure.
- When energy dissipation devices or base isolation systems are used.
- When the building site is less than 10 km to a hazardous fault, or for ground motions with near-field pulse-type characteristics, a time history analysis shall be used.

3.4.4.3 Nonlinear Static Procedure (NSP)

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be developed and subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of ASCE 41. The target displacement shall be calculated by the procedure described in ASCE 41. At least two types of lateral load pattern shall be considered, as described in ASCE 41. The pushover analysis shall be performed in two principal directions independently. Force-controlled actions shall be combined using SRSS, while deformation-controlled action shall be combined arithmetically. Due to soil properties, the embedded and underground building structures may have different behavior when they are pushed in opposite directions. In these cases the NSP shall include pushover analysis in two opposite directions (for a total of four analyses for two principal directions). When the response of the structure is not primarily in one of the principal directions, the pushover analysis shall consider non-orthogonal directions to develop a spatial envelope of capacity.

For buildings that have one or more of the following conditions, nonlinear static procedures (NSP) shall <u>not</u> be used:

- For buildings for which the effective modal mass participation factor in any one mode for each of its horizontal principal axes is not 70% or more
- If yielding of elements results in loss of regularity of the structure and significantly alters the dynamic response of the structure
- When ignoring the higher mode shapes has an important effect on the seismic response of the structure
- When the mode shapes significantly change as the elements yield
- When one of the structure’s main response is torsion
- When energy dissipation devices or base isolation systems are used

3.4.4.4 Nonlinear Dynamic Procedure (NDP)

If the Nonlinear Dynamic Procedure (NDP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load deformation characteristics of individual components and elements of the building shall be subjected to earthquake shaking represented by ground motion time histories in accordance with these design criteria. Mathematical modeling and analysis procedures shall comply with the requirements of ASCE 41.

When NDP is used, three orthogonal input ground motions shall be applied to the three-dimensional model of the structure for each set of analysis. Where the relative orientation of the ground motions cannot be determined, the ground motion shall be applied in the direction that results in the maximum structural demands.

When NDP is used, the analysis shall be performed under seven sets of ground motions, the average value of each response parameter (e.g.: force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 2.7.3.

As a minimum, the nonlinear time history analysis shall comply with the following guidelines:

- Dead and required live loads shall be applied as an initial condition.
- In case of embedded building structures, hydrostatic pressure, hydrodynamic pressure, earth pressure, and buoyancy shall be applied along with dead and required live loads. Where these loads result in reducing other structural demands, such as uplift or overturning, the analyses shall consider lower and upper bound values of these loads to compute reasonable bounding demands.
- After completion of each time history analysis, it shall be verified that those structural members, which are assumed to remain elastic, and which were modeled using elastic material properties, do in fact remain elastic and satisfy strength requirements.
- For the deformation-controlled action members the deformations shall be compared with the strain limits for each performance level as specified in this document.
- For force-controlled action members the force demand shall be resisted by capacities calculated as per ASCE 41, ACI and AISC.

3.4.4.5 Local Detailed Finite Element Model

Local detailed finite element models shall be considered as tools to better understand and validate the behavior of the structure when it cannot be obtained from the global model.

3.4.4.6 Floor Diaphragm

Mathematical models of buildings with stiff or flexible diaphragms shall account for the effects of diaphragm flexibility by modeling the diaphragm as an element with in-plane stiffness consistent with the structural characteristics of the diaphragm system.

When there is interest in the response of equipment installed on the floor diaphragm, proper modeling of the floor shall be made to capture vertical vibration modes of the floor.

3.4.4.7 Building Separation

Buildings shall be separated from adjacent structures to prevent pounding as per requirements specified in Section 2.6.10.1 of ASCE 41. Exempt conditions described in Section 2.6.10.2 of ASCE 41 shall not be permitted.

3.4.4.8 Expected Material Properties

Expected material properties shall be used in calculating the structural seismic demands. They shall conform to CSDC for concrete members and CBDS for structural steel members.

3.4.4.9 Cross Sectional Properties

Effective sectional properties shall be per Section 3.2.3.4.

3.4.4.10 Foundation Flexibility

The foundation flexibility reflecting the soil-structure interaction effects, including liquefaction, lateral spreading and other seismic phenomena, shall be considered as per Section 3.4.4.17. Pile/drilled shaft foundation stiffness shall be determined through nonlinear lateral and vertical pile analyses and shall consider group effects. If the foundation stiffness (translational and rocking) is large relative to the column or pier stiffness (i.e., foundation translational/rotational stiffness is 25 times greater than the column), then the foundation may be modeled as rigid.

Below grade structures shall be modeled as embedded structures to incorporate and simulate proper soil properties and distribution in the global model. The near field (secondary non-linear) and far field (primary non-linear) effects shall be incorporated in the model. The far field effect shall be modeled with equivalent linear elastic soil properties (stiffness, mass and damping), while the near field soil properties shall represent the yielding behavior of the soil using classic plasticity rules. Input ground motions obtained from a scattering analysis shall be applied to the ground nodes of the soil elements. The Geotechnical Data Report shall provide information relative to the scattering analysis.

At grade and above grade buildings shall be connected to the near field soil with nonlinear properties when the soil behavior is expected to be subjected to high strains near the structure. The scattered foundation motions shall be applied to the ground nodes of the soil elements.

3.4.4.11 **Boundary Conditions**

In cases where the building is connected to other structures which are not included in the model, the model shall contain appropriate elements at its boundaries to capture mass and stiffness effects of adjacent structures.

After completion of static or dynamic analysis, a check shall be performed to verify that the boundary conditions and element properties are consistent with initial modeling assumptions.

3.4.4.12 **Multidirectional Seismic Effects**

The ground motions shall be applied concurrently in two horizontal directions and vertical direction as per ASCE 41. In the demand and capacity assessment of deformation-controlled actions, simultaneous orthogonality effects shall be considered. When response spectrum analysis is used, modal combination shall be performed using the CQC approach. Spatial combination shall be performed using the SRSS technique.

3.4.4.13 **Load and Load Combinations**

Seismic loads and load combinations shall comply with the requirements of ASCE 41. For embedded and underground buildings hydrostatic pressure, hydrodynamic pressure, earth pressure and buoyancy shall be included in addition to dead load and live load. Differential settlement shall be included for buildings.

3.4.4.14 **Accidental Horizontal Torsion**

In a three-dimensional analysis, the effect of accidental torsion shall be included in the model. Accidental torsion at a story shall be calculated as the seismic story force multiplied by 5% of the horizontal dimension at the given floor level measure perpendicular to the direction of applied load. Torsion needs not be considered in buildings with flexible diaphragms.

3.4.4.15 **P-△ Effects**

Geometric nonlinearity or P-△ effects shall be incorporated in the analysis.

3.4.4.16 **Overturning**

Structures shall be designed to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces due to overturning shall be investigated for the cumulative effects of seismic forces applied at and above the level under consideration. The effects of overturning shall be evaluated at each level of the structure as specified in ASCE 41. The effects of overturning on foundations and geotechnical components shall be considered in the evaluation of foundation strength and stiffness as specified in ASCE 41.

3.4.4.17 **Soil-Structure Interaction**

For soil-structure interaction (SSI) modeling and analysis procedures, see TM 2.9.10: Geotechnical Design Guidelines.

3.4.5 Seismic Capacities of Structural Components

The component capacities shall be computed based on methods given in Chapters 5 and 6 of ASCE 41 for steel and concrete structures, respectively. However, strain limits described in the Sections 3.2.4.5 and 3.2.4.8 shall be used.

3.4.5.1 **Expected Material Properties**

Expected material properties shall be used in calculating the structural seismic capacities. They shall conform to CSDC for concrete members and CBDS for structural steel members.

3.4.5.2 **Capacity of Members with Force-Controlled Action**

Axial force, bending moment and shear capacities shall be computed in accordance with the requirement of ASCE 41.

3.4.5.3 **Capacity Protected Element Design**

In order to limit the inelastic deformations to the 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 (at least 120%) design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC for concrete members and CBDS for structural steel members.

4.0 SUMMARY AND RECOMMENDATIONS

The recommended interim seismic design criteria are summarized in Section 6.0.

5.0 SOURCE INFORMATION AND REFERENCES

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2. Newmark, N.M., “Effects of Earthquake on Dams and Embankments, Geotechnique, 15,139-160.”, 1965.
3. Goyal, A. and Chopra, A. K., “Earthquake Analysis and Response of Intake-Outlet Towers”, Report No. EERC 89-04, Earthquake Engineering Research Center, University of California, Berkeley, July 1989.
4. Hashash, Y.M.A., J.J. Hook, B. Schmidt, and J.I.-C. Yao, “Seismic design and analysis of underground structure”. Tunneling and Underground Space Technology, 2001. 16: 247-293.
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6. Penzien, J., Seismically Induced Racking of Tunnel Linings, Earthquake Engineering and Structural Dynamics, pp. 683-691, 2000.
7. AASHTO 2000, Guide Specifications for Seismic Isolation Design, 2nd Edition, GSID-2, American Association of State Highway and Transportation Officials, Washington, D.C.
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9. Anderson, D.A., Martin, G.M., Lam, Ignatius, Wang, J.N, National Cooperative Highway Research Program (NCHRP) Report 611: Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments, 2008.
10. Hashash, Y., M.A., Karina, K., Koutsoftas, D. C., and O’Riordan, N. (2010) "Seismic Design Considerations for Underground Box Structures," ASCE Conf. Proc. 384, Earth Retention Conference 3. Bellevue, Washington: pp 620-637, 2010.

6.0 DESIGN CRITERIA

6.1 GENERAL

This Technical Memorandum establishes seismic design criteria and guidance for Primary Structures which support high-speed train service as defined in Section 6.5.1.1.

6.2 CONFLICTS IN CHSTP DESIGN CRITERIA

In the event of conflicting requirements between the CHSTP Design Criteria and other standards and codes of practice, the CHSTP Design Criteria shall take precedence. For requirements which have not been included in the CHSTP Design Criteria, the order of code precedence shall be: 1) local codes; 2) U.S. National Standards; 3) others.

Where circumstances or conflicts arise in the application of CHSTP Design Criteria, the designer shall notify the Authority or delegate for guidance. The designer shall use professional judgment during design to meet current standards of practice for seismic design of structures in California.

6.3 DESIGN VARIANCES TO SEISMIC DESIGN CRITERIA

Design variances to the seismic design criteria presented in this TM shall be made following the procedure given in TM 1.1.18: Design Variance Guidelines.

Examples of performance criteria variances include:

- Exceedance of allowable strain limits for structural components that do not meet Seismic Performance Criteria.
- Exceedance of allowable deformation limits for the track and structure or Exceedance of allowable rail stresses, under an OBE event (i.e., variance to TM 2.10.10 Track-Structure Interaction)

Examples of operational criteria variances include:

- Temporary closure for repairs following an OBE event
- Extended closures for repairs following a OBE event

Variances to CHSTP performance or operational criteria must be presented according to TM 1.1.18, and subject to review and approval by the Authority or delegate.

6.4 SEISMIC ANALYSIS AND DESIGN PLAN

The designer shall develop and submit a Seismic Analysis and Design Plan to the Authority or delegate justifying each structure's General Classification, Importance Classification, Technical Classification, and analysis techniques proposed for each structure under each design earthquake for review and approval.

The plan shall discuss the pre-determined mechanism for seismic response, including the regions subject to inelastic behavior, normally limited to columns, piers, footing foundations (i.e., rocking), and abutments. The plan shall also discuss when plastic hinging of caissons, piles, or drilled shafts is expected immediately below the soil surface for soft soil conditions.

The plan shall discuss in detail each proposed analysis, indicating the analysis software to be used as well as the modeling assumptions made and the various modeling techniques to be employed. The plan shall contain commentary as to the suitability of linear versus nonlinear analysis, considering geohazards, the severity of design ground motions, induced strains in the soil and structure, and expected nonlinearities.

The Authority or delegate will review, comment upon, and ultimately provide final approval of the Seismic Analysis and Design Plan.

6.5 DESIGN CLASSIFICATIONS

6.5.1 Structural Classifications

CHST structures will provide a broad range of functions for the system. As such, consistent seismic design standards with different design objectives need to be applied to various structures. Structural classification provides the method to differentiate between different seismic design objectives for the different structural types.

6.5.1.1 General Classifications

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

Primary Structures: Primary structures are those that directly support high-speed trains, including bridges, aerial structures, tunnels, underground structures, and stations. All primary structures are subject to the design criteria contained in this technical memorandum.

The following building structures, which are essential for high-speed train service, are considered Primary structures:

- Train control, communication, and operation control facilities
- Traction power distribution facilities
- Other equipment facilities essential for high-speed train service.

High-speed train track, track support, and rail fasteners are Primary structures. Seismic design criteria for track are given in TM 2.10.10: Track-Structure Interaction.

Earthen facilities, such as embankments, fills, retaining walls, U-walls, and reinforced soil structures, which directly support high-speed trains, are Primary structures and shall be subject to seismic design criteria as given in TM 2.9.10: Geotechnical Design Guidelines.

Secondary Structures: Secondary structures are those not supporting high-speed trains. The following structures are considered Secondary structures:

- Administrative buildings
- Shop and maintenance buildings.
- Storage facilities
- Cash handling buildings
- Parking structures
- Training facilities
- Other ancillary buildings, not essential for high-speed train service.

Secondary structures shall be subject to seismic design criteria as given in TM 2.5.1: Structural Design of Surface Facilities and Buildings.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's General Classification as Primary or Secondary. The Authority or delegate shall make the final determination on the General Classification of a structure.

6.5.1.2 Importance Classification

Primary structures shall be classified according to their importance. This classification will dictate the seismic performance levels the structure is required to meet.

Important Structures: Structures that are part of a critical revenue corridor as defined by the Authority or delegate.

Ordinary Structures: All structures not designated as Important are Ordinary Structures.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's Importance Classification as Important or Ordinary. The Authority or delegate will make the final determination on the Importance Classification of a structure.

6.5.1.3 Technical Classification

Primary structures shall be further classified according to their technical complexity as it relates to design.

Complex Structures: Structures which have complex response during seismic events are considered Complex Structures. Examples of complex structural features include:

- **Irregular Geometry -** Structures that include multiple superstructure levels, variable width or bifurcating superstructures, or adjacent frames with lateral fundamental periods of vibration varying by greater than 30%.
- **Unusual Framing -** Structures that include outrigger or C-bent supports, unbalanced mass and/or stiffness distribution, or structures with concrete columns having a ratio of height to least cross sectional dimension greater than 10 if in single curvature, and 15 if in double curvature.
- **Long Span Structures -** Structures that have spans greater than 300 feet.
- **Unusual Geologic Conditions -** Structures that are subject to unusual geologic conditions, including geologic hazards outlined in TM 2.9.3: Geologic and Seismic Hazard Guidelines. This include structures founded upon:
    - soft, collapsible, or expansive soil
    - soil having moderate to high liquefaction and other seismically induced ground deformation potential
    - soil of significantly varying type over the length of the structure.

  Unusual geologic conditions shall be defined within the Geotechnical Data Report.
- **At or in close proximity to Hazardous Faults -** For guidance for structures at or in close proximity to hazardous earthquake faults (R<20 km), see TM 2.10.6: Fault Rupture Analysis and Mitigation. Structures at or in close proximity of hazardous faults shall be designed using time history analyses including consideration of vertical earthquake motions.
- **Regions of Severe Ground Motions -** Structures located at regions where the peak ground acceleration (i.e., spectral acceleration at T=0 secs.) > 0.8 g for the Maximum Considered Earthquake (MCE).

Standard Structures: Structures that are not Complex Structures and comply with the pending CHSTP Design Guidelines for Standard Aerial Structures.

Non-Standard Structures: Structures that do not meet the requirements for Complex or Standard Structures, including structures with multiple superstructure types.

As part of the Seismic Analysis and Design Plan, the designer shall make a formal statement to the Authority or delegate justifying each structure's Technical Classification as Complex, Standard, or Non-Standard. The Authority or delegate will make the final determination on the Technical Classification of a structure.

6.6 SEISMIC DESIGN POLICY

6.6.1 General

The goal of these criteria is to safeguard against loss of life, major failures, and prolonged interruption of high-speed train operations caused by structural damage due to earthquakes.

6.6.2 Seismic Performance Criteria

For structures directly supporting high-speed trains, there are three levels of Seismic Performance Criteria:

- **No Collapse Performance Level (NCL)**: Structures are able to undergo the effects of the Maximum Considered Earthquake (MCE) with no collapse. Significant damage may occur which requires extensive repair or complete replacement of some components. Occupants not on trains are able to evacuate safely. Damage and collapse due to train derailment is mitigated through containment design. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely.
- **Operability Performance Level (OPL):** Structures are able to withstand the effects of the Operating Basis Earthquake (OBE) with elastic response with no spalling, and response within structural deformations limits as given in TM 2.10.10: Track-Structure Interaction, in order to limit rail stresses and protect against derailment. No derailment occurs, trains are able to safely brake from the maximum design speed to a safe stop, passengers and operators are able to evacuate stopped trains safely. Minimal disruption of service for all systems supporting high-speed train operation. Resumption of train operation within a few hours and possibly at reduced speeds.

See Table 6-1 and Table 6-2 for performance objectives and acceptable damage for No Collapse Performance Level (NCL) and Operability Performance Level (OPL), respectively.

Table 6-1: Performance Objectives/Acceptable Damage for No Collapse Performance Level (NCL)

<table>
<tr><th>Performance Level</th><th>Performance Objectives</th><th>Acceptable Damage</th></tr>
<tr><td rowspan="3">No Collapse Performance Level (NCL)
Maximum Considered Earthquake (MCE)</td><td rowspan="3">No Collapse Performance Level (NCL):
The main objective is to limit structural damage to prevent collapse during and after a Maximum Considered Earthquake (MCE).

The performance objectives are:
1. No collapse.
2. Occupants not on trains able to evacuate safely.
3. Damage and collapse due to train derailment mitigated through containment design
4. If derailment occurs, train passengers and operators are able to evacuate derailed trains safely.
5. Extensive repairs of complete replacement of some components of the system may be required before train operation may resume.
6. For underground structures, no flooding or mud inflow.</td><td>Significant yielding of reinforcement steel or structural steel. Minor fracturing of secondary and redundant steel members or rebar is permitted, with no collapse.</td></tr>
<tr><td>Extensive cracking and spalling of concrete, but minimal loss of vertical load carrying capability</td></tr>
<tr><td>Large permanent offsets that may require extensive repairs or complete replacement before operation may resume</td></tr>
</table>

Table 6-2: Performance Objectives/Acceptable Damage for Operability Performance Level (OPL)

Performance Level	Performance Objectives	Acceptable Damage
Operability Performance Level (OPL) Operating Basis Earthquake (OBE)	**Operability Performance Level (OPL):** The main objective is for structures to withstand the effects of the Operating Basis Earthquake (OBE) elastic response with no spalling, and response within structural deformation limits as given in TM 2.10.10: Track-Structure Interaction, in order to limit rail stresses and protect against derailment. The performance objectives are: 1. No derailment, trains able to safely brake from the maximum design speed to a safe stop. 2. Occupants not on trains able to evacuate safely. 3. Train passengers and operators able to evacuate stopped trains safely. 4. Minimal disruption of service for all systems supporting high-speed train operation. 5. Resumption of train operations within a few hours and possibly at reduced speeds. 6. Safe performance in aftershocks 7. No rocking of bridge foundations 8. For underground structures, no flooding or mud inflow.	Elastic structural response, no structural damage. No spalling allowed. No track damage. Negligible permanent deformations.

6.6.3 Design Earthquakes

This criteria uses design earthquakes for which CHST facilities are to be designed to. The design earthquakes and performance levels are based upon similar criteria worldwide for high-speed trains, and current California Department of Transportation (Caltrans) standards.

Since more devastating earthquakes have a lower probability of occurrence, a probabilistic approach to defining earthquake hazard is used. The "return period" identifies the expected rate of occurrence for a level of earthquake. Additionally, deterministic methods are used to evaluate severe ground motions for the Maximum Considered Earthquake (MCE).

There are two levels of design earthquakes: the Maximum Considered Earthquake (MCE) and the Operating Basis Earthquake (OBE) defined as:

- o **Maximum Considered Earthquake (MCE**): Ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to M_{max}) of any fault in the vicinity of the structure.
- o **Operating Basis Earthquake (OBE**): Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

For more information about ground motions, including topics such as near source fling effects and the development of ground motion spectra and time histories, see TM 2.9.6: Interim Ground Motion Guidelines and TM 2.9.3: Geologic and Seismic Hazard Analysis Guidelines.

6.6.4 Hazardous Fault Crossings

TM 2.10.6: Fault Rupture Analysis and Mitigation presents the design methods and philosophies for structures at or near hazardous faults. Structures at or in close proximity of hazardous faults are classified as Complex Structures and shall be designed using time history analyses including consideration of vertical earthquake motions.

6.6.5 Seismic Design Benchmarks for 15% and 30% Design

TM 2.10.5: 15% Seismic Design Benchmarks provides guidance for 15% design. Since limited project-specific seismic and geotechnical information will be available, TM 2.10.5 gives recommended methods and assumptions to be used in order to advance the 15% design

The level of 15% seismic design is based upon a Primary structure's Technical Classification:

- 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 technically classified as "complex", Equivalent Static Analysis (ESA) for NCL performance under MCE motions is required in order to define the foundation footprints, verify structural framing feasibility, and provide preliminary construction cost estimates.

For 30% and final design, the seismic criteria defined within this TM apply.

6.7 DESIGN REFERENCES AND CODES

This Technical Memorandum uses information drawn from the following references:

1. European Standard EN 1991-2:2003 Traffic Loads on Bridges
2. European Standard EN 1990:2002 +A1: 2005 Basis of Structural Design Annex A2 Application for Bridges
3. Taiwan High Speed Rail (THSR) Corporation Volume 9 Design Specifications: Section 1: General Design Specification and Section 3: Bridge Design Specification
4. Structural Design Criteria for Devil's Slide Tunnel: Final Lining and Portals

The provisions within this Technical Memorandum shall govern the design. Provisions in the following documents shall also be considered as guidelines when sufficient criteria are not provided by this Technical Memorandum.

1. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering, 2009
2. ACI: American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318-05
3. AISC: American Institute of Steel Construction, Steel Construction Manual, 13th Edition
4. ASCE 41:Seismic Rehabilitation of Existing Structures
5. AWS D1.1/D1.1M:2008 Structural Welding Code-Steel
6. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code
7. AWS D1.8/D1.8M:2009 Structural Welding Code-Seismic Supplement
8. CBC: The 2010 California Building Code
9. California Department of Transportation (Caltrans) Bridge Design Manuals (CDBM)
   - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification 4th Edition, 2007, with California Amendments.
   - Bridge Memo to Designers Manual (CMTD)
   - Bridge Design Practices Manual (CBPD)
   - Bridge Design Aids Manual (CBDA)
   - Bridge Design Details Manual (CBDD)

- Standard Specifications
- Standard Plans
- Seismic Design Criteria ver. 1.6 (CSDC)

The design codes referenced above are current as of May, 2011. Note that since the design codes will evolve during the duration of the CHSTP, design code references are subject to change at later dates.

Design shall meet all applicable portions of the general laws and regulations of the State of California and of respective local authorities.

6.8 LAWS AND CODES

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

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

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

6.9 SEISMIC DESIGN

This Technical Memorandum (TM) establishes seismic design criteria and guidance for structures supporting high-speed train service, including but not limited to, bridges, aerial structures, tunnels, underground structures, stations, and building structures. These structures are defined as Primary structures.

Secondary structures, those not supporting, or potentially impacting, high-speed train service, shall be designed according to TM 2.5.1: Structural Design of Surface Facilities and Buildings. For seismic design criteria for earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10: Geotechnical Design Guidelines.

For MCE events, a performance (i.e., strain and deformation) based design approach shall be used.

For OBE events, a force based design approach shall be used, structures are to respond elastically.

For OBE events, TM 2.10.10: Track-Structure Interaction contains track safety and rail-structure interaction criteria concurrent with high-speed train loading. For OBE events, due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the appropriate analysis technique for the track. For the structure, an elastic analysis is appropriate.

6.10 BRIDGES AND AERIAL STRUCTURES

All bridges and aerial structures supporting high-speed train service are Primary Structures.

6.10.1 Design Codes

For MCE, current Caltrans performance based design methods and philosophies as given in Caltrans Bridge Design Manuals (CBDM) form the basis of design. Certain criteria herein exceed those of CBDM. For items not specifically addressed in this or other project specific Technical Memoranda, CBDM shall be used.

For OBE events, current Caltrans force based design methods and philosophies as given in Caltrans Bridge Design Specifications (CBDS) form the basis of design. Certain criteria herein exceed those of CBDS.

6.10.2 Seismic Design Philosophy

The seismic design philosophy differs depending upon the design earthquake.

6.10.2.1 MCE Design Philosophy

For MCE events, ductile structural response is required, whereby:

- The structure shall have a clearly defined and pre-determined mechanism for seismic response.
- Inelastic behavior shall be limited to columns, piers, footing foundations and abutments.
- The seismic detailing requirements per CSDC shall be satisfied.

Pre-determined structural components are allowed to have inelastic behavior. This provides a fusing mechanism, whereby the plastic response of the fuse limits the system demands. Other non-fusing components are designed as force-protected, with over-strength design providing a safe margin to resist the plastic demands.

The two main allowable fusing mechanisms for bridges and aerial structures are column flexural plastic hinging and foundation rocking.

In each case, the non-fusing or force-protected members shall be designed to prevent brittle failure mechanisms, such as footing shear, column to footing joint shear, column shear, tensile failure at the top of concrete footings, and unseating of girders. For design of force protected members, the column plastic moment and shear shall be used with over-strength (at least 120%) factors applied.

For flexural plastic hinging, it is generally desirable to limit plastic hinging to the columns. The location of plastic hinges shall be at points accessible for inspection and repair.

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 6.4. The capacity protected bridge superstructure shall remain essentially elastic.

Sacrificial components, such as abutment shear keys, are not subject to capacity protected response under MCE events. Stable rocking response is allowed for spread footing foundations.

Rocking is allowed during MCE events, as long as collapse is prevented.

Modeling and analysis shall conform to CBDM and CSDC.

6.10.2.2 OBE Design Philosophy

For OBE events, elastic structural response is required, whereby:

- The structure shall respond elastically under OBE response
- The track shall comply with track safety and rail-structure interaction criteria concurrent with high-speed train loading per TM 2.10.10: Track-Structure Interaction.

Rocking is not allowed for OBE events.

Verify OBE demands versus force-based capacities calculated per CBDS, with project specific amendments per Section 6.10.5.2.

6.10.2.3 Seismic Isolation

Seismic isolation may be an effective scheme to minimize damage, reduce seismic demands on substructures, and reduce foundation costs. For seismic isolation, AASHTO's Guide Specifications for Seismic Isolation Design [7] shall be used for design.

Note that seismic isolation shall contain sufficient capacity under service (i.e., braking and acceleration, wind, etc.) loads and OBE events, in order to meet criteria in TM 2.10.10: Track-Structure Interaction.

6.10.3 Seismic Demands on Structural Components

In increasing order of complexity, analysis techniques include equivalent static analysis (ESA), response spectrum analysis (RSA), equivalent linear time history analysis (ELTHA), and non-linear time history analysis (NLTHA).

The analysis technique proposed for each structure under each design earthquake shall be part of the Seismic Analysis and Design Plan.

For MCE events, the appropriate analysis technique will depend upon the site-specific conditions and complexity of the structure. The Seismic Analysis and Design Plan shall contain commentary as to the suitability of linear versus nonlinear analysis, considering geohazards, the severity of design ground motions, induced strains in the soil and structure, and expected nonlinearities

For OBE events, due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the analysis technique for the track. For the structure, an elastic analysis is appropriate.

6.10.3.1 Force Demands (F_U) for OBE

For OBE events, elastically calculated force demand, F_u, shall be determined for all structural components.

For the structure, the loading combination shall be as specified in TM 2.3.2: Structure Design Loads.

For the track, loading combinations for track safety and rail-structure interaction shall be as specified in TM 2.10.10: Track-Structure Interaction.

6.10.3.2 **Displacement Demands ($\triangle_D$) for MCE**

For MCE events, the displacement demand, $\triangle_D$, at the center of mass of the superstructure for each bent shall be determined, and compared versus the displacement capacity, $\triangle_C$.

For the structure, the loading combination shall be as specified in TM 2.3.2: Structure Design Loads.

6.10.3.3 **Vertical Earthquake Motions**

Vertical earthquake motions only apply to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6: Fault Rupture Analysis and Mitigation.

Structures at or in close proximity of hazardous faults shall be designed using time history analyses including consideration of horizontal and vertical earthquake motions.

6.10.3.4 **Effective Sectional Properties**

For MCE events, cracked bending and torsional moments of inertia for ductile and superstructure concrete members shall be per CSDC Section 5.6.

When moment-curvature analysis of concrete members is used, elemental cross sectional analysis shall be performed which considers the effects of concrete cracking, the degree of confinement and reinforcement yield and strain hardening, in accordance with CMTD and CSDC.

For structural steel sections, either moment-curvature analysis may be performed which consider the stress-strain relationship of the structural steel, or effective section properties presented derived based upon the degree of nonlinearity may be used. Seismic criteria for structural steel components are not presently incorporated in CSDC ver. 1.6., but will be incorporated in future releases of CSDC.

For OBE events, effective bending moments of inertia for concrete column members shall consider the maximum moment demand, M_a, and the cracking moment, M_{cr}, in accordance with CBDS Section 5.7.3.6.2. When using this method, the cracked moment of inertia, I_{cr}, shall be per CSDC Section 5.6. Alternatively, OBE effective sectional properties can be directly found through the use of moment-curvature analysis.

6.10.3.5 **Mass**

Both elemental and lumped mass may be used in analysis.

Translational and rotational elemental mass is based upon the mass density, length and cross sectional properties of discrete elements within the analysis model.

Translational and rotational lumped mass is based upon engineering evaluation of the structure, and often includes items modeled as rigid (i.e., pile and bent caps), or items not explicitly modeled (i.e., non-structural items).

6.10.3.6 **Expected Material Properties**

Expected material properties shall be used in calculating the structural seismic demands. They shall conform to CSDC for concrete members and CBDS for structural steel members.

6.10.3.7 **Flexural Plastic Hinging**

Where flexural plastic hinging is used as the primary seismic response mechanism of the structure, the analysis shall conform to CSDC methods and procedures.

6.10.3.8 **Assessment of Track-Structure Interaction**

For assessment of train and track-structure interaction, including requirements and load combinations which include OBE events, see TM 2.10.10: Track-Structure Interaction. For OBE events due to track-structure interaction requirements which require nonlinear fastener slippage, non-linear time history analysis (NLTHA) shall be the appropriate analysis technique for the track. For the structure, an elastic analysis is appropriate.

6.10.3.9 **Foundation Stiffness**

For caissons, pile or drilled shaft foundations, the foundation stiffness shall be considered for all types of analyses. Liquefaction, lateral spreading and other seismic phenomena as specified in Section 6.10.3.14 shall be considered.

Pile foundation stiffness shall be determined through lateral and vertical pile analysis and shall consider group effects. If the foundation stiffness (translational and rotational) is large relative to the column or pier stiffness (i.e., foundation translational/rotational stiffness is 25 times greater than the column), then the foundation may be modeled as rigid.

For shallow foundations, seismic phenomena as specified in Section 6.10.6.3 shall be considered.

6.10.3.10 **Boundary Conditions**

In cases where the structural analysis model includes only a portion of the whole structures or abutments, the model shall also contain appropriate elements at its boundaries to capture mass and stiffness effects of the adjacent structure and/or abutment.

After completion of static or dynamic analysis, a check shall be performed to verify that the boundary conditions and element properties are consistent with initial modeling assumptions.

6.10.3.11 **Continuous Welded Rail**

For structures that have continuously welded rail, with either direct fixation or ballasted track, there may be benefits to the structural performance during a seismic event provided by the rail system. The rails may serve as restrainers at the expansion joists, essentially tying adjacent frames together under seismic loading. However, this is complex behavior, which must be substantiated and validated.

Since the rail system seismic response at the expansion joists is highly nonlinear, response spectrum analysis is not appropriate. Instead, a nonlinear time-history analysis in accordance with Section 6.10.3.19, shall be performed which considers rail-structure interaction.

TM 2.10.10 Track-Structure Interaction contains details of the rail-structure interaction modeling methodology. The rail-structure interaction shall include the rails and fastening system, modeled to consider fastener slippage and rail stiffness. The capacity of the fastener connections in both shear and uplift shall be accounted for in the analysis. Without these rail-structure interaction considerations, any structural performance benefits provided by continuous welded rail shall be ignored.

6.10.3.12 **Train Mass and Live Load**

For MCE events, trains shall not be considered.

For OBE events, train live loads with impact factor and longitudinal braking forces shall be applied to the structural system, per TM 2.3.2: Structure Design Loads, as to produce the maximum effect. The number of cars to be included in the analysis will vary depending on the adjacent span lengths. Where applicable or specific analysis methods require, CHST train loads may be modeled as equivalent static distributed loads. Where equivalent distributed loads are used in the analysis, they shall account for any local or global effects to the structure due to actual concentrated axle loads.

For single track structures, when applying loading combinations for OBE events, the following train effects shall be considered simultaneously:

1. One train vertical live load + impact
2. One train longitudinal braking force
3. Mass of one train, applied at the center of mass of the train

For multiple track structures, ½ of trains potentially occupying the structure shall be considered. Where an odd number of trains potentially occupy the structure, round down to the nearest whole number of trains (example: for 3 trains, use ½(3) = 1.5 → round down to 1). When applying load combinations for OBE events, the following train effects shall be considered simultaneously:

1. ½ of the trains live load + impact
2. ½ of trains longitudinal braking force
3. Mass of ½ of the trains, applied at the center of mass of the trains

For structural design, the OBE loading combination shall be as specified in TM 2.3.2 Structure Design Loads.

For the track and when considering track-structure interaction, OBE loading combinations for track safety and rail-structure interaction shall be as specified in TM 2.10.10 Track-Structure Interaction.

6.10.3.13 P-△ Effects

For flexural plastic hinging, P-△ effects shall conform to the requirements in CSDC.

6.10.3.14 Soil Structure Interaction

For soil-structure interaction (SSI) modeling and analysis procedures, see TM 2.9.10 Geotechnical Design Guidelines.

6.10.3.15 Displacement Demand Amplification Factor

When equivalent static analysis (ESA) or response spectrum analysis (RSA) is used for MCE events, the displacement demand, $\triangle_D$, obtained shall be multiplied by an amplification factor, C, as follows:

For $T_i/T_o < 1$: $C = [0.8/ (T/T_o)] + 0.2$

For $T_i/T_o > 1$: $C = 1.0$

where:

T_i = fundamental period of structure in the longitudinal or transverse direction (including foundation stiffness)

T_o = the period centered on the peak of the longitudinal or transverse acceleration response spectrum

In order to account for the uncertainty associated with calculation of structural period for stiff structures.

6.10.3.16 Equivalent Static Analysis

Equivalent static analysis (ESA) may be used to determine earthquake demands, E:

- For MCE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure.
- For OBE events, the Force Demands, F_u

when the structure can be characterized as a simple single-degree-of-freedom (SDOF) system, and dynamic analysis will not add significantly more insight into behavior.

ESA shall apply to standard or non-standard bridge or aerial structures having no skew, and having single column piers or multiple column bents where most of the structural mass is concentrated at a single level. ESA is applicable for bridges, aerial structures, or individual frames with the following characteristics:

- Response primarily captured by the fundamental mode of vibration with uniform translation.
- Simply defined lateral force distribution (e.g. balanced spans, approximately equal bent stiffness)
- No skew

ESA shall not apply to complex bridge or aerial structures as defined in Section 6.5.1.3.

ESA earthquake demands shall be determined from horizontal spectra by either of two methods:

- Method 1: Earthquake demand, $E = (E_L^2 + E_T^2)^{1/2}$, where E_L and E_T are the responses due to longitudinal and transverse direction earthquake motions as defined below. The application of ground motion shall be along the principal axes of individual components.
- Method 2: Earthquake demand, E, by using the 100%-30% rule, for two cases:

  Case 1 : $E = 1.0E_L + 0.3E_T$

  Case 2 : $E = 0.3E_L + 1.0E_T$

For calculation of ESA earthquake demands:

Longitudinally: $E_L = C * S_a^L * W$

Transversely: $E_T = C * S_a^T * W$

Where:

C = the amplification factor, C, given in Section 6.10.3.15,

S_a^L = longitudinal acceleration response spectral value at period T_L.

T_L = fundamental period of structure in the longitudinal direction (including foundation stiffness)

S_a^T = transverse acceleration response spectral value at period T_T.

T_T = fundamental period of structure in the transverse direction (including foundation stiffness)

W = tributary dead load + superimposed dead load for MCE

W = tributary dead load + superimposed dead load + live load for OBE per Section 6.10.3.12

Effective sectional properties shall be used per Section 6.10.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE events, 5% damped response spectra shall be used to determine S_a.

For OBE events, 3% damped response spectra shall be used to determine S_a.

6.10.3.17 Response Spectrum Analysis

Response spectrum analysis (RSA) shall be used to determine earthquake demands, E:

- For MCE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when ESA does not provide an adequate estimate of the dynamic behavior.

RSA shall apply to standard or non-standard bridge or aerial structures having skewed bents or abutments ≤ 45°, and having single column piers or multiple column bents. RSA is applicable for bridges or aerial structures with the following characteristics:

- Response primarily captured by the fundamental structural mode shapes containing a minimum of 90% mass participation in the longitudinal and transverse directions.
- Skewed bents or abutments ≤ 45°,

RSA shall not apply to complex bridge or aerial structures as defined in Section 6.5.1.3.

RSA involves creating a linear, three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics. The dynamic model is used to determine the fundamental structural mode shapes for use in analysis. A sufficient number of modes shall be included to account for a minimum of 90% mass participation in the longitudinal and transverse directions. Care shall be taken to ensure 90% mass participation for long viaduct models. The designer shall examine the modes to ensure that they sufficiently capture the behavior of the structure.

A linear elastic multi-modal spectral analysis shall be performed using the appropriately damped response spectra, as given in the Geotechnical Data Report. The modal response contributions shall be combined using the complete quadratic combination (CQC) method.

For MCE events, RSA based on design spectral accelerations will likely predict forces in some elements that exceed their elastic limit, the presence of which indicates nonlinear behavior. The designer shall recognize that forces generated by RSA could vary considerably from the actual force demands on the structure. Sources of nonlinear response not captured by RSA include the effects of surrounding soil, yielding of structural members, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior.

Where there is a change in soil type along the bridge alignment or the bridge is complex, consideration shall be made to the possibility that out-of-phase ground displacements at two adjacent piers may increase the computed demand on expansion joints, rails or columns. This effect is not explicitly considered in RSA. In such cases, more sophisticated time history analyses shall be used.

Appropriate linear stiffness shall be assumed for abutments and expansion joints. Analyses shall be performed for compression models (abutments engaged, gaps between frames closed) and for tension models (abutments inactive, gaps between frames open), to obtain a maximum response envelope. If analysis results show that soil capacities are exceeded at an abutment, iterations shall be performed with decreasing soil spring constants at the abutment per CBDS and CMTD recommendations.

For calculation of differential displacements at expansion joints and for calculation of column drift, the analysis shall either explicitly compute these demands as modal scalar values or assume that the displacements and rotations combine to produce the highest or most severe demand on the structure.

RSA earthquake demands shall be determined from horizontal spectra by either of two methods:

- <u>Method 1:</u> Earthquake demand, $E = (E_L^2 + E_T^2)^{1/2}$, where E_L and E_T are the responses due to longitudinal and transverse earthquake spectra as defined below. The application of ground motion shall be along the principal axes of individual components.
- <u>Method 2:</u> Earthquake demand, E, by using the 100%-30% rule, for two cases:

  Case 1 : $E = 1.0E_L + 0.3E_T$

  Case 2 : $E = 0.3E_L + 1.0E_T$

For calculation of RSA earthquake demands:

Longitudinally: E_L = C * (RSA demands from longitudinal earthquake spectra)

Transversely: E_T = C * (RSA demands from transverse earthquake spectra)

Where:

C = the amplification factor, C, given in Section 6.10.3.15,

Effective sectional properties shall be used per Section 6.10.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 6.10.3.12.

For MCE events, 5% damped response spectra shall be used.

For OBE events, 3% damped response spectra shall be used.

6.10.3.18 **Equivalent Linear Time History Analysis**

Equivalent linear time history analysis (ELTHA) shall be used to determine earthquake demands, E:

- For MCE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when ESA or RSA provides an unrealistic estimate of the dynamic behavior.

ELTHA shall apply to standard or non-standard bridge or aerial structures having skewed bents or abutments > 45°, since the directionality of seismic motions for highly skewed structures is an important consideration.

ELTHA shall not apply to complex bridge or aerial structures as defined in Section 6.5.1.3.

ELTHA involves creating a three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics.

For MCE events, motions consistent with the 5% damped response spectra shall be used.

For OBE events, motions consistent with the 3% damped response spectra shall be used.

Rayleigh damping shall be used for ELTHA. The form of damping requires the calculation of both stiffness and mass proportional coefficients anchored at two structural frequencies, which shall envelope all important modes of structural response. The lowest structural frequency (i.e., longest period) shall be one anchor frequency, the other shall be chosen such that a minimum of 90% mass participation in the longitudinal and transverse directions are enveloped. To determine the frequency anchor at the low structural frequency, the frequency analysis shall be performed using cracked section properties and the resulting frequency reduced by 10%.

For MCE events, Rayleigh damping shall be 5%.

For OBE events, Rayleigh damping shall be 3%.

Effective sectional properties shall be used per Section 6.10.3.4.

An equivalent linear representation of foundation stiffness shall be used. Iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response).

For MCE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 6.10.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling design earthquake ground motions, as given in the Geotechnical Data Report. The motions shall consist of two-horizontal ground motion time histories, selected, scaled, and spectrally matched. The two horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motions relative to the local or global coordinate systems of the structural model.

Vertical earthquake time histories shall also be applied to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6 Fault Rupture Analysis and Mitigation. In such cases, the motions shall consist of two horizontal and one vertical ground motion time histories, selected, scaled, and spectrally matched.

When ELTHA is used, the following analyses shall be performed:

- Seven sets of ground motions, the average value of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

After completion of each ELTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

6.10.3.19 Nonlinear Time History Analysis

Nonlinear time history analysis (NLTHA) shall be used to determine earthquake demands, E:

- For MCE events, the Displacement Demand, $\triangle_D$, at the center of mass of the superstructure
- For OBE events, the Force Demands, F_u

when RSA or ELTHA provides an unrealistic estimate of the dynamic behavior, provides overly conservative demands, or where nonlinear response is critical for design.

NLTHA shall apply to complex bridge or aerial structures.

For OBE events, due to track-structure interaction requirements (per TM 2.10.10 Track-Structure Interaction) which require nonlinear fastener slippage, NLTHA shall be the analysis technique for the track, regardless of the structural classification. For the structure, ESA, RSA, or ELTHA analysis may be appropriate, dependent upon the requirements for each analysis above.

NLTHA involves creating a three-dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics. This dynamic model is used to determine the dynamic characteristics of the structure by including selected nonlinear representations of structural and foundation elements.

For MCE events, motions consistent with the 5% damped response spectra shall be used.

For OBE events, motions consistent with the 3% damped response spectra shall be used.

Rayleigh damping shall be used for NLTHA. The form of damping requires the calculation of both stiffness and mass proportional coefficients anchored at two structural frequencies, which shall envelope all important modes of structural response. The lowest structural frequency (i.e., longest period) shall be one anchor frequency, the other shall be chosen such that a minimum of 90% mass participation in the longitudinal and transverse directions are enveloped. To determine the frequency anchor at the low structural frequency, the frequency analysis shall be performed using cracked section properties and the resulting frequency reduced by 10%.

For MCE events, Rayleigh damping shall be 5%.

For OBE events, Rayleigh damping shall be 3%.

Where applicable, effective sectional properties shall be used per Section 6.10.3.4. Otherwise, cross sectional properties of concrete and steel elements with nonlinear behavior may be represented by moment-curvature relations.

Where applicable, equivalent linear representation of foundation stiffness shall be used, and iteration shall be performed until the equivalent linear foundation stiffness converges (i.e., the assumed stiffness is consistent with the calculated response). Otherwise, nonlinear representations of foundation characteristics shall be used.

For MCE events, dead and superimposed dead loads shall be applied as an initial condition.

For OBE events, in addition to dead and superimposed dead loads, live load shall be applied as an initial condition. Live loads shall be applied to produce the maximum effects in accordance with Section 6.10.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling design earthquake ground motions, as given in the Geotechnical Data Report. The motions shall consist of two horizontal ground motion time histories, selected, scaled, and spectrally matched. The two horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motion relative to the local or global coordinate systems of the structural model.

Vertical earthquake time histories shall also be applied to structures at or in close proximity to hazardous earthquake faults (R<20 km) as per TM 2.10.6 Fault Rupture Analysis and Mitigation. In such cases, the motions shall consist of two horizontal and one vertical ground motion time histories, selected, scaled, and spectrally matched. When NLTHA is used, the following analyses shall be performed:

- Seven sets of ground motions, the average value of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

After completion of each NLTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

6.10.3.20 **Rocking for MCE**

For MCE events, where rocking of the footings is used as the primary seismic response mechanism of the structure, nonlinear analysis methods are required. One acceptable method for such analysis is the most current Caltrans rocking analysis procedure, which includes the following steps:

1. Develop a relationship between the top of the column displacement and the rocking period of the footing.
2. Develop a displacement response spectrum from the design acceleration response spectrum or use the displacement response spectrum provided in the design criteria (note: the designer shall account for greater damping associated with rocking behavior as recommended in the Caltrans procedure.).
3. Begin with an initial assumed total displacement. Use a computational approach that produces a calculated total displacement.
4. If the calculated displacement equals the initial assumed displacement, convergence is reached and a stable rocking response found.
5. If the calculated displacement differs from the initial assumed displacement, then convergence not is reached. Resize the footing and iterate until convergence is reached.

When determining the rocking response of an aerial structure, consideration shall be given to possible future conditions, such as a change in depth of the soil cover above the footing or other loads that may increase or decrease the rocking response.

An alternative to the method described above, a more rigorous analysis of the rocking response shall be performed using a NLTHA.

6.10.4 Seismic Capacities of Structural Components

6.10.4.1 **Force Capacities (ΦF_N) for OBE**

For OBE design, LRFD force capacities, $\Phi F_{N,}$ for all structural components shall be found in accordance with CBDS.

6.10.4.2 **Displacement Capacity ($\triangle_C$) for MCE**

For MCE design using ESA, RSA, and ELTHA demands, the displacement capacity, $\triangle_C$, shall be determined by nonlinear static displacement capacity or "pushover analysis" as described in Section 6.10.4.3. 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. Specified capacity shall be considered to be reached when the concrete or steel strains of any primary element meets the limits specified in Sections 6.10.4.5 to 6.10.4.8.

For comparison to NLTHA demands, if a moment curvature representation of plastic hinging is used, then the curvature demands shall be converted to concrete or steel strains, and verified versus allowable strains in Sections 6.10.4.5 to 6.10.4.8.

The displacement capacity, $\triangle_C$, shall include all displacements attributed to flexibility in the foundations, bent caps, and other elastic and inelastic member responses in the system. The assumptions made to determine the displacement capacity, $\triangle_C$, shall be consistent with those used to determine the displacement demand, $\triangle_D$.

All structural members and connections shall also satisfy the capacity based performance requirements in Section 6.10.6.

6.10.4.3 **Nonlinear Static Analysis**

For MCE events, in determining the displacement capacity, $\triangle_C$, using nonlinear static pushover analysis, the following procedure shall be followed:

Dead load shall be applied as an initial step.

Incremental lateral displacements shall be applied to the system. A plastic hinge shall be assumed to form in an element when the internal moment reaches the idealized yield limit in accordance with Section 6.10.3.7. The sequence of plastic hinging through the frame system shall be tracked until an ultimate failure mode is reached. The system capacity shall then be determined in accordance with CSDC.

6.10.4.4 **Plastic Hinge Rotational Capacity**

Plastic moment capacity of ductile flexural members shall be calculated by moment-curvature (M-φ) analysis and shall conform to CSDC for concrete members and CBDS for structural steel members.

The rotational capacity of any plastic hinge is defined based on the curvature in M-φ analysis where the structural element first reaches the allowable strain limits described below.

6.10.4.5 **Strain Limits for Ductile Reinforced Concrete Members**

For MCE events, the following reinforcing steel (A706/Grade 60) allowable tensile strain limits $\varepsilon_{su}{}^{a}$) shall apply for ductile reinforced concrete members:

MCE: $\varepsilon_{su}{}^{a} \leq 2/3\ \varepsilon_{su}$

where: ε_{su} = ultimate tensile strain per CSDC.

For MCE events, the following allowable confined concrete compressive strain limits ($\varepsilon_{cu}{}^{a}$) shall apply for ductile reinforced concrete members:

MCE: $\varepsilon_{cu}{}^{a} \leq 2/3\ \varepsilon_{cu}$

where: ε_{cu} = ultimate compressive strain as computed by Mander's model for confined concrete.

6.10.4.6 **Strain Limits for Ductile Reinforced Concrete Caissons, Piles, and Drilled Shafts**

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 6.4.

For MCE events, the following reinforcing steel (A706/Grade 60) allowable tensile strain limit $\varepsilon_{su}{}^{a}$) shall apply for ductile reinforced concrete caissons, piles, and drilled shafts:

MCE: $\varepsilon_{su}{}^{a} \leq \varepsilon_{sh}$

where: ε_{sh} = tensile strain at the onset of strain hardening per CSDC

For MCE events, the following allowable confined concrete compressive strain limits ($\varepsilon_{cu}{}^{a}$) shall apply for ductile reinforced concrete caissons, piles, and drilled shafts:

MCE: $\varepsilon_{cu}{}^{a} \leq$ lesser of $1/3\ \varepsilon_{cu}$ or $1.5\ \varepsilon_{cc}$

where: ε_{cu} = ultimate compressive strain as computed by Mander's model for confined concrete.

ε_{cc} = strain at maximum concrete compressive stress as computed by Mander's model for confined concrete.

6.10.4.7 Strain Limits for Unconfined Concrete

Unconfined compressive strain limits shall 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 shall be considered unconfined.

For MCE events, the following allowable concrete unconfined compressive strain limits (ε_{cu}^{a}) apply:

MCE: $\varepsilon_{cu}^{a} = 0.004$

There are no allowable strain requirements for unconfined cover concrete.

6.10.4.8 Strain Limits for Structural Steel Elements

For MCE events, the following structural steel allowable tensile strain limits (ε_{su}^{a}) apply:
MCE: $\varepsilon_{su}^{a} \leq 2/3\ \varepsilon_{su}$
where: ε_{su} = ultimate tensile strain
Structural steel allowable compressive strain limits shall be determined based upon governing local or global buckling in accordance with CBDS, using expected material properties.

6.10.4.9 Rocking

The rocking capacity of the bridge and aerial structure piers shall be determined as per Section 6.10.3.20.

6.10.4.10 Expected Material Properties

Expected material properties shall be used in calculating structural seismic capacities, except shear. For seismic shear capacities, use nominal material properties. Expected material properties shall conform to CSDC for concrete members and CBDS for structural steel members.

6.10.4.11 Shear Capacity

Shear capacity of ductile components shall conform to CSDC for concrete members and CBDS for structural steel members.

6.10.4.12 Joint Internal Forces

For all events, continuous force transfer through the column/superstructure and column/footing joints shall conform to CSDC. These joint forces require that the joint have sufficient strength to ensure elastic behavior in the joint regions based on the capacity of the adjacent members.

6.10.5 Seismic Performance Evaluation

6.10.5.1 Rocking

For MCE events, when rocking is the primary seismic response mechanism, a stable rocking response must be provided, see Section 6.10.3.20.

For OBE events, rocking of structures is not allowed.

6.10.5.2 Force Based Design for OBE

For OBE events, the maximum force based Demand/Capacity Ratio shall be:

$$F_U / \Phi F_N \leq 1.0$$

Where:

F_U = the force demand, as defined in Section 6.10.3.1.

ΦF_N = the LRFD force capacity, as defined in Section 6.10.4.1.

in order to satisfy the OPL performance objectives specified in Section 6.6.2. See TM 2.3.2 Structure Loads for applicable load combinations.

6.10.5.3 Displacement Based Design for MCE

For MCE events, the maximum displacement Demand/Capacity Ratio shall be:

$$\triangle_D / \triangle_C \leq 1.0$$

Where:

$\triangle_D$ = the displacement demand, as defined in Section 6.10.3.2.

$\triangle_C$ = the displacement capacity, based on strain limits, as defined in Section 6.10.4.2.

in order to satisfy the NCL performance objectives specified in Section 6.6.2.

6.10.5.4 Demand versus Capacity Evaluation

Demand/capacity ratios in any three orthogonal directions may be evaluated separately for columns and footings.

For other members which carry vertical loads primarily through bending, such as superstructure members and bent caps, vertical dead and seismic D/C ratios shall be evaluated in combination with the horizontal seismic D/C ratios. In evaluating the combined D/C ratios, 1.0, 0.3, 0.3 rules shall be used for the seismic loads. The vertical dead load shall always have a factor of 1.0 applied.

When evaluating seismic loads on piles or drilled shafts, vertical and horizontal seismic loads need not be combined. However, the designer shall evaluate the piles with the column plastic moment acting about the principal axes, as well as about diagonal axes to determine the critical loading on the piles.

6.10.6 Seismic Design

All structure design shall conform to the requirements specified herein and CBDM.

6.10.6.1 Capacity Protected Element Design

In order to limit the inelastic deformations to the 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 (at least 120%) design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC for concrete members and CBDS for structural steel members.

6.10.6.2 Soil Improvement

For details of soil improvement design, see TM 2.9.10 Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding soil improvement.

6.10.6.3 Design of Shallow Foundations

For details of shallow foundation design, see TM 2.9.10 Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding design of shallow foundations.

Shallow foundations shall be designed as capacity protected structural elements under any loading or combination of loadings, including seismic loads. When designing for footing shear, column-to-footing joint shear, and moments in footings, the column plastic moment and shear shall be used with 120% overstrength factors applied.

Under OBE events, foundation rocking shall not be allowed and the soil pressure diagram shall have a compressive width of at least half of the footing width.

6.10.6.4 Design of Caisson, Pile, and Drilled Shaft Foundations

For details of caisson, pile, and drilled shaft foundation design, see TM 2.9.10 Geotechnical Design Guidelines.

The Geotechnical Data Report and Final Geotechnical Design Report shall provide information and design parameters regarding these types of foundations, such as:

- Ultimate and design load capacities in compression and tension
- Negative skin friction or down drag forces
- Resistance to lateral loads
- Group effects
- Allowable differential settlements
- Battered piles

Caisson, pile and drilled shaft foundations shall be designed as capacity protected structural elements under any loading or combination of loadings, including seismic loads. When designing for pile/drilled shaft cap shear, column-to-pile/drilled shaft cap joint shear, and moments in pile/drilled shaft cap, the column plastic moment and shear shall be used with 120% over strength factors applied.

Although plastic hinge formation is undesirable for caissons, piles or drilled shafts below the ground surface, for soft soil sites plastic hinging may be allowed immediately below the soil surface for MCE events only pending review by the Authority. Any expected plastic hinging below the ground surface must be identified in the Seismic Analysis and Design Plan as discussed in Section 6.4.

The design of piles shall be in accordance with the CBDM. The CBC special detailing requirements for seismic Zones 3 and 4 shall also be applicable to the pile design for bridges and aerial structures.

Full corrosion protection shall be provided for steel piles in the form of cathodic protection or through a corrosion allowance added to the steel section thickness.

6.10.6.5 **Battered Piles**

The use of battered piles shall, to all practical extents, be avoided. Where the use of battered piles is unavoidable, due to their relative stiffness they must carry all of the expected lateral demands, since in such scenarios vertical piles provide little lateral resistance. Where battered piles are used, displacement-strength compatibility must be considered.

Battered piles shall be designed to safely resist all imposed loadings, including resistance to crushing at the pile-pile cap interface under seismic loading. In addition, development of the pile reinforcing into the pile cap shall consider the additional significant tensile demands on these piles and potential shear failure of the piles under concurrent tensile demands. Battered piles shall not be allowed where negative skin friction is anticipated.

Battered piles shall not be farther out of plumb than one horizontal unit in three vertical units.

Where battered piles are to be used, consideration shall be given to the possibility of such battered piles encroaching on property outside the right-of-way, or interfering with existing structures or pile foundations.

6.10.6.6 **Expansion Joint and Hinge / Seat Capacity**

The detailed design of structural expansion joints shall provide free movement space for creep, shrinkage, temperature variation, braking and acceleration, and seismic response.

Under MCE response, structural expansion joints shall be verified to ensure that damaged joints will not induce changes to important structural behavior. Only local damage is acceptable.

Adequate seat length shall be provided to accommodate anticipated seismic displacements and prevent unseating of the structure. Seat width requirements are specified in CSDC for hinges and abutments. Hinge restrainers shall be designed as a secondary line of defense against unseating of girders in accordance with CSDC.

When excessive seismic displacement must be prevented, shear keys shall be provided and designed as capacity-protected elements.

Transverse shear keys shall be provided to accommodate the anticipated seismic loads without modification to the provision for thermal movement and vibration characteristics.

6.10.6.7 **Columns**

Columns shall satisfy the detailing requirements for ductile structural elements as specified in CSDC.

6.10.6.8 **Superstructures**

Superstructures shall be designed as capacity protected elements, and shall remain essentially elastic.

6.10.6.9 **Structural Joints**

Superstructure and the bent cap joints and footing joints shall conform to the requirements of CSDC.

6.11 TUNNELS AND UNDERGROUND STRUCTURES

6.11.1 General

Bored tunnels, cut-and-cover tunnels, mined tunnels, portals, U-sections, ventilation structures, and other underground structures, which directly support high-speed train service, are Primary Structures.

For seismic design criteria for earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10 Geotechnical Design Guidelines.

This document does not discuss culverts, pipelines or sewer lines, nor does it specifically discuss issues related to deep chambers such as hydropower plants, mine chambers, and protective structures. Future technical memoranda for those items are pending.

6.11.2 Design Codes

Generally, current Caltrans seismic analysis and design philosophies as stated in Caltrans Bridge Design Manuals (CBDM) form the basis of design. However, certain criteria herein exceed those of CBDM. For items not specifically addressed in this or other project specific technical memoranda, CBDM shall be used.

6.11.3 Seismic Design Philosophy

For tunnels and underground structures, the intended structural action under seismic loading is that of a Ductile Structure, whereby:

- The tunnel or underground structure shall have a clearly defined mechanism for response to seismic loads.
- Inelastic behavior shall be limited to selected regions, the remainder of the structure shall be force protected to prevent brittle failure mechanisms.

In general, the designer allows specified structural components to undergo inelastic behavior under MCE events, while force-protecting other components. The structure shall remain elastic under the OBE events.

An adequate margin of strength shall be provided between the designated load-resistance ductile mode and non-ductile failure modes. Sufficient over-strength capacity (at least 120%) shall be provided to assure the desired ductile mechanism occurs and that the undesirable non-ductile failure mechanisms are prevented from forming.

6.11.4 Seismic Demands on Structural Components

6.11.4.1 **General**

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: racking/ovaling, axial, and curvature deformations.

1. Racking/ovaling deformations primarily due to seismic waves propagating transverse to the tunnel axis.

2. Axial deformations primarily due to seismic waves along the tunnel axis.
3. Curvature deformations primarily due to seismic waves along the tunnel axis.

Appropriate modeling and analysis methods shall be used for static and seismic analyses of the tunnels and portal structures.

6.11.4.2 **Input Ground Displacements and Velocities**

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.

Ground displacements and velocities are primary considerations for the seismic design of underground structures. To assess the ground displacements and velocities induced by the design earthquakes, the effects of soil nonlinearity and soil-structure interaction shall be considered. Special problems related to the site, such as liquefaction, fault rupture and excessive settlement, shall be evaluated and taken into consideration per the Geotechnical Data Report.

Ground displacements shall be in accordance with TM 2.9.6 Interim Ground Motion Guidelines.

Soil springs, both laterally (p-y) and vertically (t-z), shall be in accordance with the Geotechnical Data Report.

For shallow buried structures in close proximity (R<20 km) to hazardous earthquake faults where seismic loadings may produce a significant inertia response, vertical effects must be considered. In such cases, the dynamic motions applied shall consist of two horizontal and one vertical ground motion time-histories, selected, scaled and spectrally matched.

The time-history analysis should include: Seven sets of ground motions, the average value of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design. After completion of each NLTHA, the designer shall verify that structural members which are modeled as elastic do remain elastic and satisfy strength requirements.

6.11.4.3 **Analysis Techniques**

The general procedure for seismic design of underground structures shall be based primarily on the ground deformation approach. During earthquakes, underground structures move together with the surrounding geologic media. The structures, therefore, shall be designed to accommodate the deformations imposed by the ground. The relative stiffness between the underground structure and surrounding soil shall be considered; the effects of soil-structure interaction shall be taken into consideration.

6.11.4.4 **Load and Load Combinations**

The seismic design and evaluation of tunnels and underground structures shall consider loading and load combinations as given in TM 2.3.2 Structure Design Loads.

6.11.4.5 **Construction Sequence**

Construction sequence including dead loads, surcharge, and potential soil arching effects shall be included as initial conditions, occurring prior to the seismic demands.

6.11.4.6 **Capacity Reduction Factors**

For evaluating the capacity protected seismic response of underground tunnels, capacity reduction factors in accordance with CBDM shall be used.

6.11.4.7 **Proximity Analysis**

If a tunnel is built in the vicinity of another tunnel, underground structure, or at-grade structure, a proximity study shall be performed. The results, conclusions, and subsequent analysis requirements of the proximity study shall be submitted to the Authority or delegate for review and comment.

6.11.4.8 Racking/Ovaling Analysis

Racking/ovaling deformations are primarily due to seismic waves propagating transverse to the tunnel axis. The deformations and strains due to these motions, which result in tunnel cross-sectional distortion, shall be evaluated by numerical methods.

As verification to numerical results, closed-form approximations of racking/ovaling demands can be found based upon the procedures outlined in [4, 5, 6, 9, 10].

6.11.4.9 Seismic Loads due to Axial and Curvature Deformations

Axial and curvature deformations are primarily due to seismic waves along the tunnel axis.

A global three-dimensional model of the tunnel shall be developed using either linear or nonlinear beam elements, as appropriate, representing the cross section of the tunnel.

The tunnel model shall be supported by either linear or nonlinear soil springs in the three orthogonal directions, as specified in the Geotechnical Data Report.

The ground motions, in accordance with TM 2.9.6 Interim Ground Motion Guidelines, shall be applied to the ground nodes of the springs.

6.11.4.10 Cross Passages and Connection Joints

The effects of stress concentration at cross-passage and connection joints to the main tunnel shall be obtained using detailed three-dimensional tunnel/soil models.

6.11.4.11 Stability

When segmental linings are used for a bored tunnel, the stability of the segments shall be verified by the use of detailed finite element models using nonlinear soil continuum and proper contact surfaces at the segment interfaces. Racking/ovaling analysis shall be performed to examine the separation of the segments and stability of the entire system.

6.11.4.12 Interface Joints

Interfaces between bored tunnel structures and the more massive structures, such as the cut-and-cover structures, stations, and ventilation/access structures, shall be designed and detailed as flexible joints to accommodate the differential movements. The design differential movements shall be determined by the designer in consultation with the Geotechnical Engineer.

6.11.5 Seismic Capacities of Structural Components

6.11.5.1 Earth Embankments, Retaining Structures

For seismic design criteria for earth supporting structures, such as earth embankments, retaining walls, and reinforced soil structures, see TM 2.9.10 Geotechnical Design Guidelines.

Information contained within the Geotechnical Data Report shall form the basis of design.

6.11.5.2 Cut-and-Cover Tunnels

For seismic design of cut-and-cover tunnels, CBDM and additional requirements in Geotechnical Data Report form the basis of design.

6.11.5.3 Tunnel Portals

Seismic design criteria for tunnel portals are under final development and approval.

Where tunnel portals consist of reinforced concrete structures, then CBDM shall form the basis of design.

6.11.5.4 Bored Tunnels

Bored tunnels include earth tunnel sections and rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining.

Seismic design criteria for bored tunnels are under final development and approval.

Where bored tunnels have reinforced concrete lining, then CBDM shall form the basis of design.

Bored tunnel sections shall be designed to sustain all the loads to which they will be subjected to, such as:

- Handling loads as determined by the transport and handling system.
- Shield thrust ram loads as determined by the shield propulsion system.
- Erection loads including external grouting loads.
- Vertical and horizontal earth pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.
- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it
- Surcharge loads due to adjacent buildings.
- Seismic demands as indicated in this TM.

Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area.

Provisions for soil-structure interaction and lateral support of surrounding ground shall be included.

6.11.5.5 **Mined Tunnels**

Mined tunnels include rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining.

Seismic design criteria for mined tunnels are under final development and approval.

Where mined tunnels have reinforced concrete lining, then CBDM shall form the basis of design.

Temporary Support Systems

Temporary support systems shall be designed to sustain all the loads to which they will be subjected, such as:

- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Additional loads due to the driving of adjacent tunnels.
- Surcharge loads due to adjacent buildings.

Cast-in-Place Liners

Cast-in-place liners shall be designed to sustain all the loads to which they will be subjected, such as:

- Handling loads as determined by the transport and handling system.
- Erection loads including external grouting loads.
- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report
- Hydrostatic pressure.
- Self-weight of the tunnel structure.

- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it.
- Surcharge loads due to adjacent buildings.
- Seismic demands as indicated in this TM.

Precast Segmental Liners

The precast segmental liners shall be designed to sustain all the loads to which they will be subjected, such as:

- Handling loads as determined by the transport and handling system.
- Shield thrust ram loads if applicable as determined by the shield propulsion system.
- Erection loads including external grouting loads.
- Vertical and horizontal rock pressures as calculated using empirical, semi-empirical, theoretical, or numerical methods, per the Geotechnical Data Report.
- Hydrostatic pressure.
- Self-weight of the tunnel structure.
- Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.
- Additional loads due to the driving of adjacent tunnels.
- Effects of tunnel breakouts at cross-passages, portals, and shafts.
- Live loads of trains moving in the tunnel or on the surface above it.
- Surcharge loads due to adjacent buildings.
- Seismic demands as indicated in this TM.

Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area.

Provisions for soil-structure interaction and lateral support of surrounding ground shall be included.

6.11.5.6 Ventilation and Access Shafts

Seismic design criteria for ventilation and access shafts are under final development and approval.

Where ventilation and access shafts have reinforced concrete lining, then CBDM shall form the basis of design.

The seismic considerations for the design of vertical shaft structures are similar to those for bored tunnels, except that racking/ovaling and axial deformations in general do not govern the design.

Consideration shall be given to the curvature strains and shear forces of the lining resulting from vertically propagating shear waves. Force and deformation demands may be considerable in cases where shafts are embedded in deep, soft soils. In addition, potential stress concentrations at the following critical locations along the shaft shall be properly assessed and designed for: (1) abrupt change of the stiffness between two adjoining geologic layers, (2) shaft/tunnel or shaft/station interfaces, and (3) shaft/surface building interfaces. Flexible connections shall be used between any two structures with different stiffness and mass in poor ground conditions.

6.12 PASSENGER STATIONS AND BUILDING STRUCTURES

6.12.1 General

All at-grade, elevated or underground passenger stations and building structures supporting high-speed train service are categorized as Primary Structures.

6.12.2 Design Codes

CBC methodology shall be used for all non-seismic related design. However, since the CBC primarily uses force-based seismic design, ASCE 41 is referenced for the performance (i.e., strain and deformation) based seismic design methodology proposed for the CHSTP.

Although ASCE 41 is a document originally issued for seismic rehabilitation of existing structures, it is pertinent here since it is very thorough and comprehensive. It is referenced in absence, at this date, of a similar performance based code for the seismic design of new building structures.

ASCE 41 is to be used to satisfy the no collapse performance level (NCL) during the Maximum Considered Earthquake (MCE).

Although the basis of the following criteria relies heavily on ASCE 41, certain criteria might exceed those of ASCE 41. If items are not specifically addressed in this or any other section of the criteria, ASCE 41 is to be used.

Passenger stations or building structures supporting high-speed train service shall withstand the effects of the Operating Basis Earthquake (OBE) within structural deformations as given in TM 2.10.10 Track-Structure Interaction, in order to limit rail stresses and protect against derailment.

6.12.3 Seismic Design Philosophy

The intended structural action under seismic loading is:

- A “weak beam strong column” philosophy shall be implemented in the design of the buildings. The plastic hinges shall form in the beams and not in the columns. Proper detailing shall be implemented to avoid any kind of nonlinearity or failure in the joints, either ductile or brittle. The formation of a plastic hinge shall take place in the beam element at not less than twice the depth of the beam away from the face of the joint by adequate detailing.
- The building shall have a clearly defined mechanism for response to seismic loads with clearly defined load path and load carrying systems.
- Each component shall be classified as primary or secondary, and each action shall be classified as deformation-controlled (ductile) or force-controlled (nonductile). The building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application to the final point of resistance. All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level.
- The detailing and proportioning requirements for full-ductility structures shall be satisfied. No brittle failure shall be allowed.

In general, the designer may allow specified structural components to undergo inelastic behavior under the MCE, while force-protecting other components. The main nonlinear mechanism is member flexural plastic hinging. The force-protected members shall be designed to prevent brittle failure mechanisms.

The structure shall remain elastic under the OBE. Active, semi-active and passive energy dissipation devices or base isolation systems are permitted. If employed, these devices and systems are a source of nonlinear mechanism in the structure, and nonlinear analysis shall be performed.

An adequate margin of strength shall be provided for nonlinear elements. Over-strength (no less than 120%) shall be provided to assure the desired nonlinear behavior and that the undesirable non-ductile failure mechanisms are prevented from forming. All structural components not pre-determined for rocking or flexural plastic hinging shall be designed to remain essentially elastic under seismic loads. Structural components can be considered essentially elastic when the induced strains exceed elastic limits, but the resulting structural damage is minor and will not reduce the ability of the structure to carry operational loads in the near and long term. For design of force protected members, the column plastic moment and shear shall be used with the appropriate over-strength factors (at least 120%) applied.

6.12.4 Seismic Demands on Structural Components

6.12.4.1 **Analysis Techniques - General**

The station or building shall be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Soil-structure interaction shall be considered in the modeling and analysis, where necessary.

Structures shall be analyzed using Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), or Nonlinear Dynamic Procedure (NDP).

Unless it is shown that the conditions and requirements for Linear Dynamic Procedure (LDP) or Nonlinear Static Procedure (NSP) can be satisfied, all structures shall be analyzed using Nonlinear Dynamic Procedure (NDP).

6.12.4.2 **Linear Dynamic Procedure (LDP)**

Linear dynamic procedure (LDP) shall be used in accordance with the requirements of ASCE 41. This can be either a response spectrum method or time-history method as applicable. Buildings shall be modeled with linear elastic stiffness and equivalent viscous damping values consistent with the behavior of the components responding at or near yield level, as defined in ASCE 41.

When response spectrum analysis is used, modal combination shall be performed using the CQC approach, while spatial combination shall be performed using the SRSS technique.

When LDP is used, the analysis shall be performed under seven sets of ground motions, the average value of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 6.6.3.

For buildings that have one or more of the following conditions, linear dynamic procedures (LDP) shall <u>not</u> be used:

- In-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.1 of ASCE 41.
- Out-of-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.2 of ASCE 41.
- Weak Story Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.3 of ASCE 41.
- Torsional Strength Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.4 of ASCE 41.
- Building structures subject to potential foundation sliding, uplift and/or separation from supporting soil (near field soil nonlinearity).
- Building structures which include components with nonlinear behavior such as, but not limited to, buckling, expansion joint closure.
- When energy dissipation devices or base isolation systems are used.
- When the building site is less than 10 km to a hazardous fault, or for ground motions with near-field pulse-type characteristics, a time history analysis shall be used.

6.12.4.3 **Nonlinear Static Procedure (NSP)**

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be developed and subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of ASCE 41. The target displacement shall be calculated by the procedure described in ASCE 41. At least two types of lateral load pattern shall be considered, as described in ASCE 41. The pushover analysis shall be performed in two principal directions independently. Force-controlled actions shall be combined using SRSS, while deformation-controlled action shall be combined arithmetically. Due to soil properties, the embedded and underground building structures may have different behavior when they are pushed in opposite directions. In these cases the NSP shall include pushover analysis in two opposite directions (for a total of four analyses for two principal directions). When the response of the structure is not primarily in one of the principal directions, the pushover analysis shall consider non-orthogonal directions to develop a spatial envelope of capacity.

For buildings that have one or more of the following conditions, nonlinear static procedures (NSP) shall not be used:

- For buildings for which the effective modal mass participation factor in any one mode for each of its horizontal principal axes is not 70% or more
- If yielding of elements results in loss of regularity of the structure and significantly alters the dynamic response of the structure
- When ignoring the higher mode shapes has an important effect on the seismic response of the structure
- When the mode shapes significantly change as the elements yield
- When one of the structure’s main response is torsion
- When energy dissipation devices or base isolation systems are used

6.12.4.4 **Nonlinear Dynamic Procedure (NDP)**

If the Nonlinear Dynamic Procedure (NDP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load deformation characteristics of individual components and elements of the building shall be subjected to earthquake shaking represented by ground motion time histories in accordance with these design criteria. Mathematical modeling and analysis procedures shall comply with the requirements of ASCE 41.

When NDP is used, three orthogonal input ground motions shall be applied to the three-dimensional model of the structure for each set of analysis. Where the relative orientation of the ground motions cannot be determined, the ground motion shall be applied in the direction that results in the maximum structural demands.

When NDP is used, the analysis shall be performed under seven sets of ground motions, the average value of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 6.6.3.

As a minimum, the nonlinear time history analysis shall comply with the following guidelines:

- Dead and required live loads shall be applied as an initial condition.
- In case of embedded building structures, hydrostatic pressure, hydrodynamic pressure, earth pressure, and buoyancy shall be applied along with dead and required live loads. Where these loads result in reducing other structural demands, such as uplift or overturning, the analyses shall consider lower and upper bound values of these loads to compute reasonable bounding demands.
- After completion of each time history analysis, it shall be verified that those structural members, which are assumed to remain elastic, and which were modeled using elastic material properties, do in fact remain elastic and satisfy strength requirements.
- For the deformation-controlled action members the deformations shall be compared with the strain limits for each performance level as specified in this document.
- For force-controlled action members the force demand shall be resisted by capacities calculated as per ASCE 41, ACI and AISC.

6.12.4.5 Local Detailed Finite Element Model

Local detailed finite element models shall be considered as tools to better understand and validate the behavior of the structure when it cannot be obtained from the global model.

6.12.4.6 Floor Diaphragm

Mathematical models of buildings with stiff or flexible diaphragms shall account for the effects of diaphragm flexibility by modeling the diaphragm as an element with in-plane stiffness consistent with the structural characteristics of the diaphragm system.

When there is interest in the response of equipment installed on the floor diaphragm, proper modeling of the floor shall be made to capture vertical vibration modes of the floor.

6.12.4.7 Building Separation

Buildings shall be separated from adjacent structures to prevent pounding as per requirements specified in Section 2.6.10.1 of ASCE 41. Exempt conditions described in Section 2.6.10.2 of ASCE 41 shall not be permitted.

6.12.4.8 Expected Material Properties

Expected material properties shall be used in calculating the structural seismic demands. They shall conform to CSDC for concrete members and CBDS for structural steel members.

6.12.4.9 Cross Sectional Properties

Effective sectional properties shall be per Section 6.10.3.4.

6.12.4.10 Foundation Flexibility

The foundation flexibility reflecting the soil-structure interaction effects, including liquefaction, lateral spreading and other seismic phenomena, shall be considered as per Section 6.12.4.17. Pile/drilled shaft foundation stiffness shall be determined through nonlinear lateral and vertical pile analyses and shall consider group effects. If the foundation stiffness (translational and rocking) is large relative to the column or pier stiffness (i.e., foundation translational/rotational stiffness is 25 times greater than the column), then the foundation may be modeled as rigid.

Below grade structures shall be modeled as embedded structures to incorporate and simulate proper soil properties and distribution in the global model. The near field (secondary non-linear) and far field (primary non-linear) effects shall be incorporated in the model. The far field effect shall be modeled with equivalent linear elastic soil properties (stiffness, mass and damping), while the near field soil properties shall represent the yielding behavior of the soil using classic plasticity rules. Input ground motions obtained from a scattering analysis shall be applied to the ground nodes of the soil elements. The Geotechnical Data Report shall provide information relative to the scattering analysis.

At grade and above grade buildings shall be connected to the near field soil with nonlinear properties when the soil behavior is expected to be subjected to high strains near the structure. The scattered foundation motions shall be applied to the ground nodes of the soil elements.

6.12.4.11 Boundary Conditions

In cases where the building is connected to other structures which are not included in the model, the model shall contain appropriate elements at its boundaries to capture mass and stiffness effects of adjacent structures.

After completion of static or dynamic analysis, a check shall be performed to verify that the boundary conditions and element properties are consistent with initial modeling assumptions.

6.12.4.12 Multidirectional Seismic Effects

The ground motions shall be applied concurrently in two horizontal directions and vertical direction as per ASCE 41. In the demand and capacity assessment of deformation-controlled actions, simultaneous orthogonality effects shall be considered. When response spectrum analysis is used, modal combination shall be performed using the CQC approach. Spatial combination shall be performed using the SRSS technique.

6.12.4.13 Load and Load Combinations

Seismic loads and load combinations shall comply with the requirements of ASCE 41. For embedded and underground buildings hydrostatic pressure, hydrodynamic pressure, earth pressure and buoyancy shall be included in addition to dead load and live load. Differential settlement shall be included for buildings.

6.12.4.14 Accidental Horizontal Torsion

In a three-dimensional analysis, the effect of accidental torsion shall be included in the model. Accidental torsion at a story shall be calculated as the seismic story force multiplied by 5% of the horizontal dimension at the given floor level measure perpendicular to the direction of applied load. Torsion needs not be considered in buildings with flexible diaphragms.

6.12.4.15 P-△ Effects

Geometric nonlinearity or P-△ effects shall be incorporated in the analysis.

6.12.4.16 Overturning

Structures shall be designed to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces due to overturning shall be investigated for the cumulative effects of seismic forces applied at and above the level under consideration. The effects of overturning shall be evaluated at each level of the structure as specified in ASCE 41. The effects of overturning on foundations and geotechnical components shall be considered in the evaluation of foundation strength and stiffness as specified in ASCE 41.

6.12.4.17 Soil-Structure Interaction

For soil-structure interaction (SSI) modeling and analysis procedures, see TM 2.9.10 Geotechnical Design Guidelines.

6.12.5 Seismic Capacities of Structural Components

The component capacities shall be computed based on methods given in Chapters 5 and 6 of ASCE 41 for steel and concrete structures, respectively. However, strain limits described in the Sections 6.10.4.5 and 6.10.4.8 shall be used.

6.12.5.1 Expected Material Properties

Expected material properties shall be used in calculating the structural seismic capacities. They shall conform to CSDC for concrete members and CBDS for structural steel members.

6.12.5.2 Capacity of Members with Force-Controlled Action

Axial force, bending moment and shear capacities shall be computed in accordance with the requirement of ASCE 41.

6.12.5.3 Capacity Protected Element Design

In order to limit the inelastic deformations to the 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 (at least 120%) design factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC for concrete members and CBDS for structural steel members.

6.13 SOURCE INFORMATION AND REFERENCES

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